APPENDIX 'A'

GEOTECHNICAL REPORT

Replacement of the Dublin Avenue Bridge over Omand's Creek

Geotechnical Report

March 29, 2019

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1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) was retained by the City of Winnipeg (City) to complete the preliminary design, detailed design, and construction administration services for the replacement of the Dublin Avenue Bridge over Omand's Creek project in Winnipeg, Manitoba. The purpose of this report is to outline the geotechnical engineering services that have been performed for the replacement structure. The geotechnical engineering report for the roadway replacement/repair will be presented in a separate report.

The work that has been performed includes the following:

- Review of available existing geotechnical information within the project area.
- Perform a geotechnical testhole drilling and sampling program near the existing bridge structure to identify the existing soil conditions. Two testholes were drilled on August 3, 2018 and included the installation of one vibrating wire piezometer to monitor the groundwater level with time.
- Conduct a laboratory testing program on select soil samples to help establish the engineering properties relevant to the bridge replacement assessment including moisture contents, particle size analyses and Atterberg limit tests.
- Review of alternate foundation types for the proposed crossing including the development of geotechnical design capacities within the Limit State Design framework.
- Review of slope stability conditions for alternate crossing options.
- Preparation of a summary report (this report) outlining the existing soil conditions and providing recommendations for the various aspects of the proposed crossing.

2.0 PROJECT SITE AND BACKGROUND INFORMATION

The Replacement of the Dublin Avenue Bridge Over Omand's Creek project includes the removal of the existing bridge and design and construction of a replacement structure. The existing Dublin Avenue bridge is located approximately 400 m west of the intersection of Notre Dame Avenue and Dublin Avenue in Winnipeg, Manitoba. The existing bridge consists of two east bound lanes and two west bound lanes, and spans approximately 25 m. The existing bridge consists of a timber and concrete structure which is nearing the end of its useful life. As such, Stantec was retained to provide preliminary design, detailed design, and construction administration services for the replacement crossing structure. An approximate length of 1300 m of roadway replacement/repair works is also included in Stantec's scope of work. This report, however, is limited to the bridge replacement structure. The geotechnical assessment for the proposed roadway replacement will be submitted as a separate report.

For the replacement structure, Stantec investigated the feasibility of several alternatives with the results outlined in the memorandum titled "Dublin Bridge Replacement – Review of Viable Crossing Alternatives", dated August 23, 2018. Following completion of the feasibility review, two alternates have been selected to advance to the preliminary design stage as follows:

1. Box Culvert Option – The proposed box culvert option consists of a three-cell box culvert with approximate cell dimensions of 2.7 m wide and 2.61 m height. A 90 mm thick asphalt overlay (with waterproofing membrane) is also included directly on top of the culvert. The box culvert underside elevation matches the existing structure soffit elevation (top Elev. 234.240 m). The general configuration of this option is shown on the "Proposed Culvert Structure – General Arrangement – S-101" drawing in **Appendix B**.

2. Bridge Option – The proposed bridge option consists of a semi-integral solid concrete slab bridge. The deck thickness is proposed to be 500 mm with a 90 mm thick asphalt overlay (with waterproofing membrane), and the superstructure soffit is set at top Elev. 234.292 m. The general configuration of this option is shown on the "Proposed Bridge Structure – General Arrangement – S-201" drawing in **Appendix B**.

The geotechnical works described in this report have been performed to identify the existing soil conditions at the site and develop geotechnical design parameters so that each currently considered option can be advanced to preliminary design. This report may also be used to advance the preliminary design to detailed design, following selection of a preferred alternative by the City of Winnipeg.

3.0 GEOTECHNICAL INVESTIGATION

A geotechnical investigation program has been completed for this project to identify and assess the existing soil stratigraphy at the site and estimate representative soil shear strength parameters for use in design. The geotechnical investigation program for this project consisted of a drilling and sampling program as well as a laboratory testing program.

3.1 DRILLING AND SOIL SAMPLING

The geotechnical drilling and sampling program was conducted on August 3, 2018, with drilling services provided by Paddock Drilling Ltd. under continuous supervision by Stantec geotechnical personnel. A total of two testholes were advanced using an Acker MP5 track-mounted drill rig. Both testholes were advanced until auger refusal which was encountered at depths of 15.4 m and 14.0 m for testholes TH18-01 and TH18-02, respectively. The testhole locations are shown on Drawing G-101 located in **Appendix B**. As shown, testhole TH18-01 is located approximately 15 m south of the existing west abutment on the upper bank of Omand's Creek. Testhole TH18-02 is located approximately 12 m north of the existing west abutment on the mid-bank of Omand's Creek. The detailed testhole records are included in **Appendix C**. Also included in **Appendix C** is a summary sheet outlining the symbols and terms used on the testhole records.

The drilling program consisted of advancing 125 mm diameter solid stem augers through the overburden materials to power auger refusal. Overburden samples were retrieved directly off the auger flights at approximate 0.75 to 1.5 m intervals and at locations of material changes. Field pocket torvane testing was performed on cohesive samples at approximate 0.75 to 1.5 m intervals. Attempts were made to advance 35 mm inside diameter split spoon samples within the underlying till materials to obtain Standard Penetration Test (SPT) "N" values. Upon completion of testhole drilling, the testholes were examined for evidence of soil sloughing and groundwater conditions then backfilled with auger cuttings and bentonite pellets. All samples were visually inspected in the field for material types and consistency and transferred to our Winnipeg, Manitoba, materials testing laboratory for further inspection and testing.

3.2 SITE GEOMETRY

The existing geometry of the site may be summarized as follows:

- The existing Omand's Creek banks south of the bridge (i.e. downstream) have been previously remediated and naturalized. Both east and west banks are approximately 2.5 m in height and have a side slope of approximately 4H:1V. The creek bottom appears to be approximately 4 to 5 m wide. Small diameter rockfill riprap has been provided along parts of the channel as part of the remediation/naturalization works. Refer to Photo 1 and Photo 2 in **Appendix D**.
- The existing east bank of Omand's Creek north of the bridge (i.e. upstream) is approximately 2.5 m in height and has a relatively steep side slope of approximately 1.5H:1V. The bank appears to slope from an existing building and parking lot area to the creek bottom. Refer to Photo 3 and Photo 4 in **Appendix D**.
- The existing west bank of Omand's Creek north of the bridge (i.e. upstream) is approximately 2.5 m in height. The bank has a side slope of approximately 2.5H:1V from the top of bank to a relatively lower flat bank over a height of 1.5 to 2 m. The lower bank area is approximately 7 m wide and slopes at approximately 2.5H:1V to the

creek bottom. The creek bottom is approximately 2 to 3 m wide. Refer to Photo 5, Photo 6, and Photo 7 in **Appendix D**.

- The creek banks north of the bridge (i.e. upstream) contained heavy vegetation growth, making existing bank performance observations difficult (Photo 8 in **Appendix D**).

3.3 INSTRUMENTATION

One vibrating wire piezometer was installed directly adjacent to testhole TH18-01. Piezometer ID 100D1800939 was installed on August 3, 2018, with the piezometer tip at geodetic Elev. 224.97 m within clay material (approximately 8.9 m below existing grade). The piezometer was installed to monitor groundwater conditions for use in the slope stability analyses.

3.4 LABORATORY TESTING

A laboratory testing program was performed on select representative soil samples from the geotechnical investigation program to measure the relevant engineering properties of the subsurface materials. Diagnostic testing included moisture content testing on all samples, particle size analyses (AST D442) and Atterberg limits tests (ASTM D4318). The results of the laboratory tests have been included on the testhole records in **Appendix C** and presented on the laboratory results summary sheets in **Appendix F**.

4.0 INVESTIGATION RESULTS

4.1 SOIL STRATIGRAPHY

The overall stratigraphic conditions of the site have been inferred based upon the results of the geotechnical investigation program. In general, the soil stratigraphy at the testhole locations consists of a surficial layer of topsoil, overlying clay fill/clay, overlying silt, overlying clay, overlying silt till.

A description of the soil stratigraphy is provided below. The detailed testhole records are provided in **Appendix C**, which also includes the symbols and terms used on the testhole records.

4.1.1 Topsoil

A thin layer of topsoil was encountered at the surface of both testholes. The topsoil was brown to black in colour, with rootlets, some sand, and ranged in thickness between approximately 0.1 m to 0.3 m.

4.1.2 Clay Fill

A layer of clay fill was encountered beneath the topsoil layer in testhole TH18-01. The clay fill layer was extended to a depth of 0.9 m (approximate Elev. 233 m). The clay fill layer was brown in colour and was with sand, with gravel, and with silt. The moisture content of the clay fill was 19%

4.1.3 Fat Clay

A layer of fat clay was encountered underlying the clay fill in testhole TH18-01 and underlying the topsoil layer in testhole TH18-02. The fat clay layer was generally observed to be brown to grey in colour and contained trace to with silt, trace to some sand, and trace silt inclusions. A layer of silt was encountered within the fat clay layer in both testholes (described in Section 4.1.4 below). Based upon field pocket torvane testing, the undrained shear strength of the fat clay ranged from approximately 9 kPa to 96 kPa (average of 31 kPa), classifying the fat clay as very soft to stiff in consistency. The clay layer generally became softer with depth. The fat clay layer extended to depths of 11.6 m (Elev. 222.3 m) and 11.9 m (Elev. 220.2 m) for testholes TH18-01 and TH18-02, respectively. The moisture content of the fat clay ranged from 30% to 63%, with an approximate average of 51%. Based upon Atterberg limit testing, the Liquid Limit of the fat clay layer ranged between 67% to 95% and the plasticity index ranged from 47% to 67%, classifying this material as high plasticity (i.e. fat).

4.1.4 Silt

A layer of silt was encountered within the fat clay layer in both testholes. The silt layer was observed to be loose, tan in colour, and contained with to some fine-grained sand, trace to some clay, and trace gravel. The silt was encountered at a depth of 1.7 m in testhole TH18-01 and at a depth of 0.6 m in testhole TH18-02. The thickness of the silt layer ranged from approximately 1.4 m to 0.6 m and extended to elevations of 231.3 m and 230.9 m for testholes TH18-01 and TH18-02, respectively. The undrained shear strength of the silt layer was estimated to be 10 kPa based upon a single field pocket torvane test. The moisture content of the silt layer ranged from 22% to 23%.

4.1.5 Sand/Silty Sand

Sand/silty sand was encountered beneath the fat clay layer in testhole TH18-01 at a depth of 11.6 m (Elev. 222.3 m). The sand/silty sand were generally observed to be grey to brown in colour, and contained fine to coarse grained sand, trace gravel, and occasional cobbles. Standard Penetration Tests (SPTs) completed within the sand layer show uncorrected "N" values ranging from 5 blows per 300 mm of penetration to 20 blows per 300 mm of penetration classify the material as loose to compact in relative density. The moisture content of the sand/silty sand ranged from 20% to 22%.

4.1.6 Silt Till

Glacial silt till was inferred to be encountered beneath the sand layer in testhole TH18-01 and was encountered beneath the fat clay layer in testhole TH18-02. There was no sample recovery below the sand layer in testhole TH18-01, however it was inferred to be silt till as a noticeable change in drilling was observed and frequent cobbles were encountered. The silt till layer was inferred to be encountered at a depth of 14.6 m (Elev. 219.3 m) in testhole TH18-01 and was encountered at a depth of 11.9 m (Elev. 220.2 m) in testhole TH18-02. The silt till layer was grey in colour, loose to very dense, of low plasticity to non-plastic, and contained with to some clay, with sand, and some gravel. Power auger refusal was reached in the silt till layer within both testholes at depths ranging from 14 to 15.4 m (Elev. 218.5 m to Elev. 218.1 m). Standard Penetration Tests (SPT) completed within the silt till layer show uncorrected SPT "N" values ranging from 23 blows per 300 mm to SPT "refusal" (classified as more than 50 blows for less than 150 mm of penetration. The moisture content of the silt till layer ranged from 15% to 26%.

4.2 GROUNDWATER AND SLOUGHING CONDITIONS

Groundwater and soil sloughing conditions were recorded upon completion of drilling each testhole and are summarized in Table 1 below.

Table 1 - Observed Groundwater and Soil Sloughing Conditions

Testhole No.	Groundwater Seepage	Observed Depth of Groundwater Seepage (m)	Depth of Groundwater Upon Completion of Drilling (m)	Observed Depth of Soil Sloughing (m)
TH18-01	Moderate	11.6 m	8.5 m	12.2 m
TH18-02	Moderate to Heavy	12.2 m	8.2 m	No sloughing

The vibrating wire piezometer installed adjacent to testhole TH18-01 has been monitored a total of five (5) times following installation (as of the time of preparation of this report), with the piezometer monitoring data included in **Appendix E**. The monitored piezometer data correlates to an approximate short-term groundwater depth between approximately 2.6 m to 2.9 m below ground surface (approximate Elev. 231.0 m to Elev. 231.3 m).

4.3 LABORATORY TEST RESULTS

A laboratory testing program was performed on selected soil samples and consisted of moisture content tests on all samples (ASTM D2216), particle size analyses (ASTM D433), and Atterberg limits (ASTM D4318). The moisture

content results can be seen on the testhole records included within **Appendix C**. A summary of the particle size analyses, and Atterberg limits testing are shown on Table 2 and Table 3, respectively. The laboratory summary sheets for all testing performed have been included within **Appendix F**.

Table 2 - Particle Size Analysis Results

Testhole No.	Sample Depth	Soil Type	Particle Size			
			Gravel 75 to 4.75 mm	Sand <4.75 to 0.075 mm	Silt <0.075 to 0.002 mm	Clay <0.002 mm
TH18-01	2.7 m	Silt	0%	7.0%	79.0%	11.4%
TH18-01	3.8 m	Clay	0.2%	0.5%	9.4%	80.0%
TH18-01	6.9 m	Clay	0%	0.9%	21.5%	67.7%
TH18-02	2.7 m	Clay	0%	0.9%	10.1%	80.5%
TH18-02	8.4 m	Clay	0.2%	8.2%	27.5%	53.4%

Table 3 - Atterberg Limits Test Results

Testhole No.	Sample Depth	Soil Type	Liquid Limit	Plastic Limit	Plasticity Index	Activity
TH18-01	2.3 m	Silt	19	17	2	N/A
TH18-01	3.8 m	Clay	95	28	67	0.84
TH18-01	6.9 m	Clay	89	25	64	0.95
TH18-02	2.7 m	Clay	90	27	63	0.78
TH18-02	8.4 m	Clay	67	20	47	0.88

5.0 FOUNDATION REVIEW

Based on the results obtained during the geotechnical field and laboratory programs, suitable foundations for the proposed crossing may consist of the following:

1. Box Culvert Option – shallow foundation, driven steel piles, driven precast concrete piles, or bored cast-in-place concrete piles.

2. Bridge Option – driven steel piles, driven precast concrete piles, or bored cast-in-place concrete piles.

Each foundation alternative is discussed in the following sections.

5.1 LIMIT STATES DESIGN

In accordance with the 2014 Canadian Highway Bridge Design Code (CHBDC), the use of Limit States Design (LSD) is required for the design of bridges and their structural components including foundations. The limit states of LSD are classified into two groups; the Ultimate Limit States (ULS) and the Serviceability Limit States (SLS).

The ULS case is primarily concerned with the collapse mechanisms for the structure and hence, safety. For foundation design, the ultimate limit state consists of:

- Exceeding the load-carrying capacity of the foundation;
- Sliding;
- Uplift;
- Large deformation of foundation, leading to an ultimate limit state being induced in the superstructure or building;
- Overturning; and
- Loss of overall stability.

The factored resistance at the ULS is the ultimate geotechnical resistance multiplied by the appropriate resistance factor as outlined in the CHBDC.

The Serviceability Limit State (SLS) case considers mechanisms that restrict or constrain the intended use of occupancy of the structure. These are typically associated with movements that interrupt or hinder the purpose of the structure. For foundation design, the serviceability limit state consists of:

- Excessive movements; and
- Unacceptable vibrations.

The SLS case is addressed by determining the maximum available resistance to keep the foundation under service loads within tolerable limits as provided by the structural engineer. Unfactored permanent and transitory loads are used for calculating total deformation in non-cohesive soils. Unfactored permanent loads and appropriate portions of transitory loads are used for the initial and time-dependent final deformations of cohesive soils. Therefore, the

foundation loads and serviceability tolerances must be known to properly determine the SLS resistance values. In cases where tolerable movements are not provided by the structural engineer, the tolerable limit of total settlement for foundations subject to compression is typically assumed to be 25 mm.

5.2 FROST PENETRATION DEPTH

The estimated depths of frost penetration have been estimated for a range of annual air freezing indices identified in Table 4 below. The mean annual freezing index is based upon published climate normal from Environment Canada between 1981 and 2010 for the Winnipeg James Richardson International Airport. The ten year return annual freezing index was calculated using the mean value based on recommendations outlined in the Canadian Foundation Engineering Manual 4th Edition (CFEM, 2006). The fifty year return annual freezing index was inferred from Figure K-5 of the National Building Code of Canada (2010) Commentary document.

Table 4 - Estimated Frost Penetration Depth

Parameter	Period		
	Mean	10-Year Return	50-Year Return
Annual Air Freezing Index (°C-Days)	1705	2299	2375
Estimated Frost Penetration – Concrete, non-snow cover (m)	1.6	1.9	1.9
Estimated Frost Penetration – Snow Cover (m)	1.2	1.4	1.4

For foundation design considerations, the CFEM recommends using the ten-year return annual freezing index to predict frost penetration. For this site, a potential frost penetration depth of approximately 1.4 to 1.9 may therefore be used in design. For purposes of this report, a potential frost penetration depth of 1.9 m has been assumed because the culvert interior will likely not be filled with snow.

The United States Army Corp of Engineers (USACE) frost design soil classification system is a widely used system that places soils is one of four categories. The categories (or Frost Groups), from F1 to F4, reflect an increase in frost susceptibility and decrease in strength during thaw. The silt and clay materials at this site (i.e. upper 2 m) is categorized as Frost Group F3 and therefore are considered highly frost susceptible.

5.3 ADFREEZE

Frozen soil in contact with unheated foundation elements can develop an adfreeze bond which can result in uplift forces on the foundations. The CFEM recommends adfreeze bond stresses for fine grained soils as follows:

- 65 kPa for fine grained soils frozen to wood or concrete; and
- 100 kPa for fine grained soils frozen to steel.

This adfreeze stress should be applied to the perimeter of a pile or any other foundation element to a depth of 1.9 m measured from final grade. The uplift forces from adfreeze stresses are resisted by the permanent dead load of the structure plus the uplift resistance of the foundation elements.

5.4 FOUNDATION DESIGN PARAMETERS

The following subsections outline geotechnical design parameters for alternate foundation types including shallow and deep foundation options.

5.4.1 Box Culvert as Footing

The base of the box culvert may be designed as a shallow strip footing founded on the native clay material. The bearing capacity for a particular footing is dependent on several factors including the footing width, the footing length, burial depth, groundwater elevations, and soil strengths.

It is our understanding that the proposed box culvert consists of a three-cell structure with a footprint of approximately 9.3 m by 24.6 m, with the base of the box culvert to be at or near the existing creek bottom elevation at approximate Elev. 231 m. The design creek bottom is Elev. 231.4 m, therefore the base of the box culvert will be buried approximately 0.4 m.

For the design of footings, the depth that is influenced by the footing loading is typically between B to 2B below the footing, where B is equal to the footing width. Geotechnical design parameters are therefore selected based upon material strengths within this influence depth. For footings on cohesive soils, capacities are typically derived based upon the undrained shear strength of the soil.

For this site, the width of the proposed box culvert is 9.3 m. The soil stratigraphy below the base of the culvert (approx. Elev. 231 m) consists of approximately 12 m of fat clay overlying silt till. As such, the entire clay depth will be influenced by the footing loading.

Based upon the available field pocket torvane data below Elev. 231, the undrained shear strength of the clay material ranges from 55 to 9 kPa, with an average of 25 kPa. Based upon an average undrained shear strength of 25 kPa for the native clay material, Table 5 below lists net bearing capacity values that may be used for design purposes.

Table 5 - Box Culvert Footing Bearing Capacity

Footing Size	Footing Depth	ULS Capacity (kPa)	SLS Capacity (kPa)
9.3 m width footing	0.4 m	70	55

The capacities provided on Table 4 above have been developed based on the following assumptions:

- A geotechnical resistance factor of 0.5 has been used to determine the ULS capacity as outlined in the CHBDC. The SLS capacity has been based upon limiting potential settlements of the culvert to less than 25 mm.
- The box culvert is placed at a depth of 0.4 m below the design bottom of Omand's Creek (design channel bottom at approximate Elev. 231.4 m).

- The groundwater level has been assumed to be at the culvert invert.
- The average undrained shear strength of the clay layer below Elev. 231.0 m is estimated to be 25 kPa based upon available data.
- A 600 mm thick layer of crushed, free-draining granular material is provided beneath the box culvert. The granular material should consist of hard crushed rockfill, free from organic material and fines, with a recommended gradation between approximate 12 mm to 50 mm particle diameters. The granular material should be placed in maximum 150 mm lifts and be roller compacted. The granular layer should extend a minimum of 1.0 m beyond all edges of the box culvert.
- Prior to the placement of the granular material, all surficial topsoil, organics, clay fill, silt, or other deleterious material must be removed down to a competent native inorganic clay subgrade. The existing clay subgrade should be compacted prior to any fill placement to a minimum of 98% of the material's SPMDD.

5.4.1.1 Potential Frost Heave

There is the potential for frozen soil conditions to develop up to approximately 1.9 m below the underside of the box culvert. As such, seasonal heave and settlement may occur due to freezing and thaw action. The values of seasonal heave are difficult to determine and are dependent on several factors including, but not limited to, meteorological conditions, snow cover, and depth of water in the creek. Potential frost movements have been estimated based upon an approximate 5 to 8% expansion of the available water content within the soil pore spaces within the frost penetration depth (i.e. approximate expansion volume of water to ice). Based on the proposed final configuration of the box culvert, it is estimated that potential frost movements of 30 mm to 80 mm are possible. These estimates are based upon the box culvert being placed on a 600 mm thick layer of roller compacted, crushed, non-frost susceptible granular material. The minimum frost movement estimate of 30 mm is based on a 1.4 m frost penetration depth and 5% water expansion, and the maximum estimate of 80 mm is based upon the 1.9 m frost penetration depth and 8% water expansion.

If this amount of potential seasonal heave is undesirable, insulation can be provided to limit the frost penetration depth or additional recommendations can be provided relating to removal of in-situ material (in addition to the currently proposed 600 mm) and replacement with non-frost susceptible material. Alternatively, the box culvert could be placed on deep foundations such as driven steel H-piles to mitigate potential frost related movements.

If insulation is the preferred method by the City to mitigate potential frost beneath the culvert, a minimum 200 mm thick layer of extruded polystyrene rigid board insulation (e.g. STYROFOAM™ HIGHLOAD) should be provided. The insulation should be placed beneath the entire footprint of the box culvert and extend a minimum of 2.44 m beyond all edges of the box culvert. A minimum 300 mm thick layer of roller compacted, crushed free-draining granular material should be provided beneath the insulation. All insulation should have a minimum 200 mm of cover to prevent against insulation degradation.

The above recommended insulation requirements have been developed based upon the methodology as outlined within the CFEM (2006) and the document titled "Design of Insulated Foundations" which was published in the American Society of Civil Engineers (ASCE) Journal of the Soil Mechanics and Foundation Division and authored by Robinsky and Bespflug (1973). The methods presented by Robinsky and Bespflug are considered to be the current industry state of practice for design of insulated foundations. It is important to recognize that an assumption of the Robinsky and Bespflug method for unheated structures is that the granular layer beneath the insulation is allowed to

freeze. As the box culvert at the Dublin site will experience perpetually wet conditions, it is possible that some frost related movements may occur under the insulated scenario. It is therefore extremely important that the granular fill material be non-frost susceptible, such as free-draining rockfill with approximate range of diameters between 12 mm and 50 mm and no fines content. The "50 mm MAX" crushed sub-base material as outlined within the City of Winnipeg standard construction specification CW3110-R19 would be generally suitable.

In addition, intermittent traffic loading has the potential to locally increase porewater pressures directly beneath the culvert, especially during spring or other wet events. Increased porewater pressures would cause a decrease in effective strengths of the underlying soils potentially resulting in settlement.

5.4.2 Shallow Strip or Spread Footings

Wing walls or other ancillary concrete structures may be constructed using a conventional shallow strip or spread footings founded on the native clay material at a minimum depth of 2 m below grade. Based upon an average undrained shear strength of 25 kPa, Table 6 below lists appropriate bearing capacity values for various footing sizes.

Table 6 - Shallow Footings Bearing Capacity

Footing Size	Footing Depth	ULS Capacity (kPa)	SLS Capacity (kPa)
1 m by 1 m	2 m	115	115
2 m by 2 m	2 m	115	115
1 m by 10 m	2 m	100	100
2 m by 10 m	2 m	100	100

The capacities provided on Table 6 above have been developed based on the following assumptions:

- A geotechnical resistance factor of 0.5 has been used to determine the ULS capacity as outlined in the CHBDC. The SLS capacity has been based upon limiting potential settlements of the culvert to less than 25 mm.
- The footing base is located at a minimum depth of 2 m below grade. The average undrained shear strength of the native clay material below approximate 2 m depth is estimated to be approximately 25 kPa.
- To minimize the potential volume change of the clay subgrade, it is recommended that a lean mix concrete working slab be constructed directly on the compacted clay subgrade.
- Prior to the placement of the granular fill, all surficial topsoil, organics, clay fill, silt, or other deleterious material must be removed down to a competent native inorganic clay subgrade. The existing clay subgrade should be compacted prior to any fill placement to a minimum of 98% of the Standard Proctor Maximum Dry Density (SPMDD).

It is noted that the bearing capacity values provided on Table 6 above are much greater than the box culvert bearing capacity values provided in Table 5. The main factors for the differences in bearing capacity are the shape of the footing (i.e. width and length), the footing burial depth, and the influence depth of the footing loading.

It is Stantec's recommendation that all shallow strip or spread footings should be placed at least 2 m below final grade (i.e. below the frost penetration depth) to mitigate potential frost related movements.

5.4.3 Driven Steel H-Piles

A potential foundation type to support either the box culvert option or the abutments for the bridge option consists of driven steel H-piles.

Based upon Stantec's experience, it has been observed that the capacities of straight shaft steel H-piles driven to practical refusal on dense till materials within the Winnipeg area are generally within the range of 50% to 70% of the structural capacity of the steel member. To account for any uncertainties associated with the actual hardness of the till, the axial capacity of a driven steel H-pile at this site is assumed to be 60% of the structural capacity of the steel, therefore:

$Q_u=0.6\ A_t\ f'_y$

Where,

Q_u = ultimate pile capacity (kN)

A_t = cross sectional area of the pile tip (m^2)

f'_y = yield strength of the pile steel (kPa)

The axial compressive pile capacity for a driven steel HP310x110 pile has been developed and is shown on Table 7 below.

Table 7 - Driven Steel H-Pile Capacity

Pile Size	Axial Compressive Resistance at ULS, Φ=0.5 (kN)	Axial Tensile Resistance at ULS (excluding frost adhesion resistance), Φ=0.3 (kN)	Refusal Criteria (three consecutive sets)
HP310x110	1480	155	12 blows/25 mm

For driven steel H-piles, SLS conditions generally do not govern design as the loads required to induce 25 mm of movement exceed those at ULS. Vertical settlements of steel H-piles driven to refusal are expected to be negligible and limited to the elastic compression of the steel pile material.

The estimated capacities provided above have been based on the following assumptions:

1. The steel piles are driven to refusal within the underlying till material.
2. A typical yield stress of 350 MPa has been used for the steel material.
3. A geotechnical resistance factor (Φ) or 0.5 for axial compression and 0.3 for tension has been used as per the CHBDC. To utilize the pile capacity as given on Table 7, Pile Driving Analyzer (PDA) tests **must** be completed on at least 5% of the total number of piles driven at this site.

4. Driven steel piles should have a minimum pile spacing of 2.5 pile diameters measured centre-to-centre within pile groups. Provided that pile spacing is a minimum of 2.5 pile diameters, pile heave is monitored, and piles are re-driven when pile heave is observed, no reduction in pile capacity would be required due to group action effects.

5. The recommended hammer energy to be used in driving steel piles should be a minimum of 49 kJ per blow. The recommended hammer energy may vary depending on the energy transfer ratio. As a guideline, pile driving can be terminated using a refusal criteria of 12 blows/25 mm for three consecutive sets. To help minimize the damage to the end of the pile during the driving process, a driving shoe should be installed at the end of each pile. The driving shoe should not extend beyond the pile perimeter tip area of the steel H-pile to limit disturbance of the soils during installation of the pile.

If the capacities given on Table 7 are insufficient, an increased size of driven steel H-pile can be reviewed.

5.4.4 Driven Precast Prestressed Concrete Hexagonal Piles

Driven precast prestressed concrete hexagonal (PPCH) piles are considered to be a suitable foundation option for either the proposed culvert option or the abutments for the bridge option. Geotechnical capacities at ULS for precast concrete piles driven to refusal within the underlying till in the Winnipeg region are reasonably well established and are given on Table 8 below.

Table 8 - Driven Precast Concrete Pile Capacities

Pile Size (mm)	Axial Compressive Resistance at ULS, Φ=0.5 (kN)	Axial Tensile Resistance at ULS (excluding frost adhesion resistance), Φ=0.3 (kN)	Refusal Criteria (three consecutive sets)
305	550	105	5 blows/25 mm
356	750	125	8 blows/25 mm
406	1000	140	12 blows/25 mm

For driven precast concrete piles, SLS conditions generally do not govern the design since the loads required to induce 25 mm of movement (i.e. the typical SLS criteria) exceed those at ULS. Vertical settlements of precast concrete hex piles driven to refusal are expected to be negligible.

The estimated capacities provided above have been based on the following assumptions:

1. PPCH piles are driven to refusal within the underlying till material.

2. A geotechnical resistance factor (Φ) or 0.5 for axial compression and 0.3 for tension has been used as per the CHBDC. To utilize the pile capacity as given on Table 8, Pile Driving Analyzer (PDA) tests must be completed on at least 5% of the total number of piles driven at this site.

3. Driven PPCH piles should have a minimum pile spacing of 2.5 pile diameters measured centre-to-centre within pile groups. Provided that pile spacing is a minimum of 2.5 pile diameters, pile heave is monitored, and piles are re-driven when pile heave is observed, no reduction in pile capacity would be required due to group action effects.

4. The pile capacities have been based upon using a pile driving hammer having a minimum rated energy of 40 kJ and a pile refusal criteria between 5 blows/25 mm to 12 blows/25 mm (as outlined in Table 8) for three consecutive sets. If an alternate hammer is proposed, Stantec should be contacted to review the refusal criteria.

5.4.5 Cast-In-Place Concrete Piles

Cast-in-place concrete piles are considered to be a suitable foundation option to support either the box culvert option or the bridge option. Cast-in-place concrete piles do not require pile driving therefore reducing the risk of potential damages to existing infrastructure during pile installation.

The ultimate capacity of a straight shaft cast-in-place concrete pile has been developed based upon the methodology as outlined within the CFEM. As per the CFEM, the ultimate capacity for cast-in-place concrete piles within cohesive soils (i.e. fat clay layer) and cohesionless soils (i.e. underlying sand or till layers) can be estimated by the following:

$$Q_u = C_1 L_1 \alpha S_u + C_2 L_2 \beta \sigma'_v + A_t N_t \sigma'_t$$

Where,

Q_u = ultimate axial compressive geotechnical capacity (kN)

C_1 = perimeter of pile within the fat clay layer (m)

L_1 = length of pile within the fat clay layer (m)

α = adhesion coefficient ranging from 0.5 to 1.0 as per Figure 18.1 of the CFEM

S_u = undrained shear strength of the fat clay layer (kPa)

C_2 = perimeter of pile within the underlying silt till layer (m)

L_2 = length of pile within the underlying silt till layer (m)

β = a combined shaft resistance factor as outlined in Table 18.1 of the CFEM

σ'_v = vertical effective stress adjacent to the pile at depth, z (kPa)

A_t = area of the pile tip (m^2)

N_t = toe bearing capacity factor as per Table 18.2 of the CFEM

σ'_t = vertical effective stress adjacent to the pile tip (kPa)

The estimated axial compressive and tensile capacities for a 457 mm diameter cast-in-place concrete pile installed 1 m within the underlying till at this site have been developed and are provided on Table 9 below.

Table 9 - Cast-In-Place Concrete Pile Capacity

Pile Diameter	Approximate Pile Length (m)	Axial Compressive Resistance at ULS, Φ = 0.4 (kN)	Axial Tensile Resistance at ULS (excluding frost adhesion resistance), Φ=0.3 (kN)
457 mm	13	650	130

The capacities provided in Table 9 have been based on the following assumptions:

- The base of the pile hole has been assumed to be mechanically cleaned and free of loose drill cuttings to ensure full end bearing resistance will be developed.
- An undrained shear strength of the fat clay layer ranging from 55 kPa to 5 kPa and decreasing with depth.
- Adhesion coefficient, α, ranging from 0.7 to 1.0 which increases with decreasing undrained shear strength.
- A combined shaft resistance factor, β, of 0.4 within the underlying silt till layer.
- A toe bearing capacity factor, N_t, of 80 within the underlying silt till layer.
- To be conservative, a groundwater level has been assumed to be at the location of the existing ground surface.
- Geotechnical resistance factors (Φ) or 0.4 for axial compression and 0.3 for axial tension (excluding frost adhesion) have been used as per the CHBDC.

If the capacities given on Table 9 are insufficient, 457 mm diameter cast-in-place concrete piles may be installed deeper within the underlying till, socketed into the underlying bedrock, or an increased pile diameter can be reviewed. Note that the depth to intact bedrock is not known, and additional investigations may need to be performed.

5.4.6 Uplift Resistance Due to Frost Adhesion

It is increasingly becoming the general state of practice in Canada to use a geotechnical resistance factor of 0.7 to 1.0 for uplift resistance due to frost adhesion, where previously a resistance factor of 0.3 was used. The general methodology of using this increased resistance factor is as follows:

- The loads associated with adfreeze uplift forces are reasonably well defined and are considered conservative.
- The forces associated with adfreeze bonds and the associated required geotechnical resistance is generally more applicable to the SLS case (i.e. to limit the upwards movement of the piles).
- The adfreeze forces are generally not applied to the structure all at once and develop over the course of the winter season.

For this site, a resistance factor of 0.7 was used in calculating the frost adfreeze resistance forces. The factored geotechnical frost adhesion resistance is given below on Table 10. The total resistance to the seasonal frost adhesion forces will be the total dead weight of the structure plus the applicable uplift resistance per pile given on

Table 10. The frost adhesion resistance presented below has been assumed to be developed within the existing overburden materials and does not include the resistance within the embankment fills.

Table 10 - Frost Adhesion Resistance

Pile Type	Pile Size	Factored Frost Adhesion Resistance, Φ=0.7 (kN)
Driven Steel Pile	HP310x110	350
Driven Precast Concrete Pile	305 mm	250
	356 mm	290
	406 mm	330
Cast-In-Place Concrete Pile	457 mm	300

For the use of deep foundations (i.e. piles) at this site, adfreeze forces along the upper approximate 2 m of the pile length are possible. The possible frost adfreeze forces for each pile type is as given on Table 11 below. As stated previously, the adfreeze forces would be resisted by the values given in Table 10 (factored frost adhesion resistance), the buoyant weight of the pile and the permanent axial vertical load.

Table 11 - Potential Adfreeze Forces

Pile type	Pile Size	Adfreeze Bond Stress (kPa)	Adfreeze Force (kN)
Driven Steel H-Pile	HP310x110	100	200
Driven Precast Concrete Pile	305 mm	65	140
	356 mm	65	160
	406 mm	65	185
Cast-in-place Concrete Pile	457 mm	65	190

5.5 PILE INSTALLATION CONSIDERATIONS

5.5.1 Driven Pile Installation

The pile hammer energies and refusal criteria that have been provided in Section 5.4.3 for driven steel H-piles and Section 5.4.4 for driven PPCH piles are considered to be general guidelines. Stantec should be contacted once the driving hammer and pile types/sizes have been selected to refine the pile driving criteria using a wave equation analyses program.

As previously stated, in order to utilize the capacities as provided in Table 7 for driven steel H-piles and Table 8 for PPCH piles, Pile Driving Analyzer (PDA) tests must be completed on a minimum of one pile per abutment. These tests should be performed at the end of initial drive (EOID) of the pile and at the beginning of restrike (BOR) of the pile to ensure that the piles reach and maintain the specified capacity. The BOR test should be performed after a period of 24 to 72 hours. It is recommended that Case Pile Wave Analysis Program (CAPWAP) analyses are performed in conjunction with PDA tests during pile installation to confirm the design capacities.

5.5.2 Cast-In-Place Pile Installation

It is anticipated that temporary steel sleeving will be required during the installation of cast-in-place concrete piles to minimize the potential for soil sloughing and groundwater infiltration within the pile holes. Efforts should be made to dewater the pile holes prior to concrete placement. If the pile holes cannot be dewatered prior to concrete placement, tremie techniques may be used.

As noted in Section 4.1.5, a sand layer was encountered overlying the silt till layer in testhole TH18-01. Excavation of the pile shafts at this depth may experience blow-up of this confined sand material. There are limited measures available to mitigate this phenomenon and there is the potential that the pile shafts will need to be cleaned out several times.

5.5.3 Pile Installation Monitoring

For cast-in-place concrete piles, it is recommended that Stantec geotechnical staff be present on-site full-time to record the actual depth of each pile and ensure proper installation conditions.

For driven piles, it is recommended that monitoring of all pile installations is performed by an experienced Stantec inspector to verity that the piles are installed in accordance with the design assumptions and that the driving criteria are satisfied. For each pile, a complete driving record in terms of the number of blows per 300 mm of penetration should be recorded by the inspector and reviewed by the designer.

If existing infrastructure or utilities are near the proposed pile installations and sensitive to vibrations, vibration monitoring should be performed to monitor and mitigate potential damages.

6.0 SLOPE STABILITY REVIEW

A slope stability review has been performed to determine the existing stability conditions of the Omand's Creek banks and to determine whether slope stability improvements are required for the proposed development of the site.

The slope stability analysis was performed using the software SLOPE/W, developed by GeoSlope International Inc. (Calgary, AB). For the slope stability analysis, the Morgenstern-Price generalized limit equilibrium solution with constant interslice force inclination has been used. The Morgenstern-Price method simultaneously solves for force and moment equilibrium, and is considered to be the current industry state of practice. The computer model investigates a large number of potential failure surfaces and depending on the method of analysis used can present the results in the form of contours of computed factors of safety against sliding.

Stability of a slope is typically generalized as a ratio of the forces that resist failure divided by the forces that drive failure. This unitless fraction is called a Factor of Safety (FS). Factors of safety that are unity (1.0) or less indicate that driving forces exceed resisting forces and from a geotechnical engineering perspective the slope has failed or is highly unstable. Due to the natural variability of soils and groundwater levels that can affect the driving and resisting forces unpredictably, the geotechnical engineering industry typically requires a minimum FS of 1.5 for long term steady state (i.e. normal groundwater levels) scenarios and 1.3 for short term transient (i.e. construction) scenarios.

A total of four (4) cross-sections were reviewed as part of the slope stability analysis. The reviewed cross-sectional information has been obtained from the survey data (contours shown on Drawing G-101) and the conceptual crossing plans including in **Appendix B**. The location of the cross-sections are shown on Drawing G-101 (**Appendix B**) and can be described as follows:

1. Existing Omand's Creek bank approximately 25 m south of the existing bridge structure (i.e. downstream)
   i. The overall bank height is approximately 2.5 to 3 m.
   ii. Previous restoration works have been performed at this location to remove contaminated soils. The side slopes have been previously flattened to approximately 4H:1V and small diameter rockfill riprap has been placed as part of naturalization works.

2. Existing Omand's Creek bank approximately 10 m north of the existing bridge structure (i.e. upstream)
   i. The overall bank height is approximately 2.5 m.
   ii. This west side of this cross-section currently has a side slope of approximately 2H:1V to 2.5H:1V and slopes from the top of bank to a relatively flat lower bank. The lower bank area is approximately 7 m wide and slopes at approximate 2.5H:1V to the creek bottom.
   iii. The east side of this cross-section has a relatively steep side slope of approximately 1.5H:1V.
   iv. This cross-section is currently located on private property. It is our understanding, however, that this location will be developed as part of the proposed replacement crossing.

3. Proposed replacement bridge structure.
   i. The proposed replacement bridge structure will include minimal abutment backfill (<1 m) and will have a headslope lined with a 500 mm thick rockfill riprap layer at 3H:1V.

4. Proposed replacement box culvert structure.

i. The proposed driving surface will be placed directly on the top of the box culvert.
ii. It is our understanding that the creek banks immediately upstream and downstream of the proposed culvert will be flattened to approximately 3H:1V and will be lined with a 500 mm rockfill riprap layer.

Two distinct groundwater scenarios were reviewed for the slope stability analyses; a "normal" groundwater/creek level both at Elev. 232 m, and a "critical" groundwater/creek level consisting of a fully saturated ground surface combined with a dry creek bed to represent a rapid drawdown scenario following a flood event. This "critical" case is considered very conservative as significant time would likely be required to achieve full bank saturation during a flood event and the rapid drawdown conditions are considered a very low probability occurrence.

6.1 SHEAR STRENGTH PARAMETERS

The selection of soil shear strength parameters is critical to any slope stability assessment, as the established factor of safety for a given slip surface is a function of the available shear resistance along the slip surface. The selection of representative soil shear strength parameters for the slope stability analyses have consisted of utilizing the available testhole information, laboratory testing results and typical values published in literature. The estimated soil shear strength parameters used in the slope stability analyses are provided in Table 12 below.

Table 12 - Slope Stability Shear Strength Parameters

Material	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle (º)
Firm Clay	17	2	20
Silt	18	1	25
Soft Clay	17	2	16
Rockfill Riprap	20	0	35
Granular fill (abutment fill)	20	0	30

6.2 SLOPE STABILITY RESULTS

The results of the slope stability analyses are summarized on Table 13 below, with all outputs included in **Appendix G**.

Table 13 - Slope Stability Results

Conditions	Cross Section	Slip Surface	GWL	FS	Figure No
Existing	1 - 25 m south (downstream)	West Bank	Normal	1.86	1
			Critical	1.19	2
		East Bank	Normal	1.96	3
			Critical	1.26	4
	2 - 10 m north (upstream)	West Bank	Normal	2.13	5
			Critical	1.33	6
		East Bank	Normal	1.19	7
			Critical	0.69	8
Crossing Replacement	3 - Bridge	West Bank/East Bank	Normal	1.54	9
			Critical	1.21	10
	4 - Box Culvert	West Bank/East Bank	Normal	1.53	11
			Critical	1.22	12

The results of the slope stability analyses shown on Table 13 can be concluded as follows:

<u>25 m south (downstream)</u>

The target factors of safety have been met for the "normal" cases (Figure 1 and Figure 3 in **Appendix G**) but have not met the target factors of safety for the "critical" cases (Figure 2 and Figure 4 in **Appendix G**).

Although the estimated factors of safety are below the target values, it is Stantec's opinion that the creek banks in this area are not at risk of significant future movements. The "critical" case represents the worst-cast situation of a post-flood rapid drawdown. As the likelihood of a full rapid drawdown scenario is considered to be low, the estimated factors of safety are only slightly below the target values, and no critical infrastructure exists within this reach, it is Stantec's opinion that the estimated factors of safety are adequate.

It is worthwhile to note that previous bank restoration work has also been completed along this reach of Omand's Creek. In Stantec's opinion, additional stabilization works are not required south of the proposed crossing.

<u>10 m north (upstream)</u>

For the west bank, the target factors of safety have been met for the "normal" and "critical" cases on Figure 5 and Figure 6 in **Appendix G**.

For the east bank, the target factors of safety <u>have not been met</u> for either the "normal" or "critical" cases as shown on Figure 7 and Figure 8 in **Appendix G**. The existing side slopes appear to be too steep, and this section of creek bank is considered at risk of slope movements under post-flood rapid drawdown conditions.

It is our understanding that this location is currently private property, however it will become developed as part of the proposed crossing replacement works. It is important to note that there is a parking lot and a building immediately adjacent (east) of the creek bank, therefore traditional slope flattening cannot be performed. It is Stantec's

recommendation to realign Omand's Creek to the west, within the "lower flat area" of the west bank, to allow for flattening of the east bank side slope to achieve adequate slope stability factors of safety. The realigned creek and flattened slopes are recommended to match the existing geometry of Omand's Creek south of the crossing (i.e. 4H:1V).

Cross Replacement – Bridge and Box Culvert

All target factors of safety have been met for the "normal" cases for both replacement crossing options (Figure 9 and Figure 11 in **Appendix G**), and are slightly below the target factors of safety for the "critical" case (Figure 10 and Figure 12 in **Appendix G**). The factors of safety that are slightly below the target are considered acceptable due to the following:

- The "critical" case represents a worst-case situation of a rapid drawdown scenario
- The proposed crossing is located adjacent to two parking lots (i.e. paved structures), therefore the likelihood of full bank saturation is low
- Potential stabilizing effects of the bridge structure foundation/box culvert have not been incorporated into the model
- The factors of safety are only slightly below the target value of 1.3 (i.e. 0.09 below the target for Figure 10 and 0.08 below the target for Figure 12).

As per the slope flattening mentioned above, the existing Omand's Creek must be realigned slightly to the west to allow for the final proposed crossing configuration. The final side slopes at the replacement crossing structure are recommended to be at a minimum of 3H:1V at the location of the replacement crossing. The side slopes should transition to the recommended 4H:1V side slope north of the structure as discussed previously.

If the creek can be realigned and the side slopes can be flattened as currently proposed, no additional creek bank stabilization measures are considered to be required north of the proposed crossing.

7.0 GENERAL CONSTRUCTION RECOMMENDATIONS

7.1 SUBSURFACE STRUCTURES AND LATERAL EARTH PRESSURES

Headwalls, wingwalls, or any other below grade walls for the proposed replacement crossing must be designed to resist lateral earth pressures. Estimates of the geotechnical parameters for use in the design of substructure walls are provided on Table 14.

Table 14 - Geotechnical Parameters for Design of Substructure Walls

Parameter	Granular Backfill
Unit weight of compacted granular backfill, γ (kN/m^3)	20
Angle of internal friction for compacted granular backfill, φ (°)	35
Coefficient of At-Rest Lateral Earth Pressure, K_o	0.43

The proposed subsurface structures should be designed to resist lateral soil pressures from compacted granular backfill materials. Due to the rigidity of the proposed concrete subsurface structures (i.e. structurally connected members), it is recommended that the walls be designed for static earth pressures assuming an "at-rest" pressure distribution over the full height of the embedded wall, where K_o is the "at-rest" earth pressure coefficient.

For a depth of up to approximately 2.5 m, the lateral earth pressure should be designed for a uniform pressure of 20 kPa, resulting from stresses from the compaction of the granular backfill. At depths exceeding 2.5 m, it is sufficient to assume that the lateral earth pressure increases linearly with depth.

The recommended lateral earth pressure distributions against the walls of proposed concrete substructures are presented on Figure 1 in **Appendix H** with the design total lateral soil pressure calculated as follows:

$P_t = P_c + P_o$

Where,
P_t =design total lateral earth pressure (kPa per m width)
P_c = pressure from compaction activities (kPa per m width)
P_o =earth pressure from at-rest soil pressure (kPa per m width)

The above expression assumes the subsurface walls will be drained and there will be no buildup of hydrostatic pressure on the walls. A 0.3 m (minimum) wide layer of free-draining granular material or an approved drainage layer product must be provided adjacent to the below grade walls and a subsurface drainage system must be provided at the base of the walls to prevent the buildup of hydrostatic pressure. Well-graded granular fill is not recommended as a drainage layer due to the reduced flow rates within this type of material. If a drainage layer is not provided adjacent to subsurface walls, the full hydrostatic pressure should be added to the above lateral earth pressure equation and applied over the buried depth of the subsurface wall.

7.2 FOUNDATION CONCRETE

Based upon Stantec's experience in the Winnipeg, MB area, the class of exposure for concrete in contact with the local subsurface soils may be severe (class exposure S-2 in CSA A23.1-14, Table 3). The requirements for concrete exposed to severe sulphate attack are provided on Table 15 below.

Table 15 - Mix Requirements for Foundation Concrete

Parameter	Design Requirement (CSA A23.1-14)
Class of exposure	S-2
Minimum specified compressive strength	32 MPa at 56 days
Air content	Air content to be determined depending on exposure to freezing and thawing and maximum sizes of coarse aggregate, as per CSA A23.1-14, Table 2 and Table 4
Maximum water-to-cementing ratio	0.45
Cementing materials type	HS or HSb

All concrete in contact with the native soils at this site should meet the requirements outlined in Table 15 above.

7.3 TEMPORARY EXCAVATIONS AND DEWATERING

Temporary excavations may be required for the construction of the replacement crossing. The stability of temporary excavations is a function of several factors including the total time the excavation is exposed, soil moisture conditions, soil type and consistency, and the contractor's operations. It is the responsibility of the contractor to maintain safe and stable side slopes or design and provide shoring during construction. As a guideline, temporary excavations may be sloped at 1 horizontal to 1 vertical within the native clay materials. If temporary shoring is to be used, it should be designed by a registered professional engineer who has experience with excavation shoring. The design of the shoring should take into consideration lateral surface pressures from construction equipment that may be near the excavation. All excavations must comply with Manitoba Workplace Safety and Health regulations.

Groundwater seepage and infiltration may occur, and pumps should be available to remove water that collects in excavations. Surface water flow from Omand's Creek should be prevented from entering any excavation. Excavated materials must be stockpiled at a sufficient distance away from the edge of any excavation so that local stability conditions are not jeopardized.

8.0 SUMMARY

Stantec was retained by the City of Winnipeg to complete the preliminary design, detailed design, and construction administration services for the replacement of the Dublin Avenue Bridge over Omand's Creek project in Winnipeg, Manitoba. This report was prepared to outline the geotechnical engineering design services that have been performed for this assignment which includes:

- Review of available existing geotechnical information within the project area;
- A testhole drilling, sampling and instrumentation program near the existing bridge structure and a laboratory testing program to identify the existing soil conditions.
- Review of alternate foundation types for the proposed crossing including the development of geotechnical design capacities within the Limit State Design framework;
- Review of slope stability conditions for alternate crossing options;
- Preparation of a summary report (this report) outlining the existing soil conditions and providing recommendations for the various aspects of the proposed crossing.

The geotechnical investigation was completed on August 3, 2018 and included advancing two testholes to power auger refusal. Testhole TH18-01 was advanced approximately 15 m south of the existing west abutment to a depth of 15.4 m (Elev. 218.5 m). A vibrating wire piezometer was installed immediately adjacent to TH18-01 with a tip elevation at 8.9 m depth (Elev. 225.0 m). Testhole TH18-02 was advanced approximately 12 m north of the existing west abutment to a depth to 14.0 m (Elev. 218.1 m). The general stratigraphy encountered within the testholes consisted of approximately 0.1 to 0.3 of topsoil, overlying 0.3 to 1.1 m of fat clay/clay fill, overlying 0.6 to 1.2 m of silt, overlying 10 m of fat clay, overlying glacial silt till. A 2.4 m thick sand layer was encountered in testhole TH18-01 at a depth of 11.6 m.

At the time of preparation of this report, the vibrating wire piezometer installed near TH18-01 has been monitored a total of five (5) times since installation. The monitored data correlates to an approximate groundwater elevation of approximately Elev. 231 m.

For the proposed replacement crossing, two options have been reviewed consisting of a box culvert option and a solid slab bridge option. Several foundation alternatives were reviewed for each option and geotechnical design parameters were developed in accordance with the Limit States Design framework. The foundation alternatives reviewed for each crossing option are as shown on the table below.

Option	Shallow Foundations	Driven Steel H-Piles	Driven Precast Concrete Piles	Cast-in-place Concrete Piles
Box Culvert	x	x	x	x
Bridge		x	x	x

Each foundation alternative is considered to be feasible, however several risk items were noted for each foundation alternative:

- For shallow foundations related to the box culvert option, there are concerns related to potential heave and settlement caused by seasonal frost movements within the relatively soft, high plastic clay soils. To reduce the magnitude of potential frost heave, a 600 mm layer of non-frost susceptible material was recommended to be placed beneath the box culvert. Alternatively, insulation with a reduced granular thickness or a fully piled foundation may be implemented. Additional movements may also occur under wet periods caused by traffic surcharge loading. The construction of shallow foundations for the box culvert option may also encounter dewatering/seepage issues from Omand's Creek during excavation of the foundation.
- For driven piles, vibrations during pile installation may affect existing adjacent utilities or infrastructure, and vibration monitoring should likely be performed during pile driving.
- For cast-in-place concrete piles, there is the potential for groundwater infiltration and soil sloughing within the pile holes which may require full depth steel sleeving. A sand layer was also observed overlying the till layer in testhole TH18-01 which may cause difficulties during excavation (i.e. pile base "blow-up").

Slope stability analyses were completed to review the existing conditions of the Omand's Creek banks and to review the final slope stability conditions of the proposed crossing structure. The estimated factors of safety for the existing slope stability conditions south of the crossing structure have been determined to be adequate and therefore no slope stability improvements are required within this area. The existing slope stability conditions of the west bank, north of the crossing structure, also meet or exceed the minimum target factors of safety. The existing slope stability conditions of the east bank, north of the crossing, do not meet the required target factors of safety. It was recommended to re-align Omand's Creek to the west to allow for slope flattening of the east bank. The realigned creek and flattened side slopes should match the existing geometry of Omand's Creek south of the crossing.

9.0 CLOSURE

This report has been prepared for the sole benefit of the City of Winnipeg and its agents, and may not be used by any third party without the express written consent of Stantec Consulting Ltd. Any use, which a third party makes of this report, is the responsibility of such third party. Use of this report is subject to the Statement of General Conditions provided in **Appendix A**. It is the responsibility of the City of Winnipeg who is identified as "the Client" within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec Consulting Ltd. should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying or unexpected site conditions
- Planning, design or construction

We trust the above information meets with your present requirements. Should you have any questions or require further information, please contact us. This report has been prepared by Rhett Bonham, P.Eng. and reviewed by Thomas Crilly, M.Sc., P.Eng.

We appreciate the opportunity to assist you in this project.

Appendix A
STATEMENT OF GENERAL CONDITIONS

Appendix A: Statement of Terms and Conditions

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Stantec's present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Stantec is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

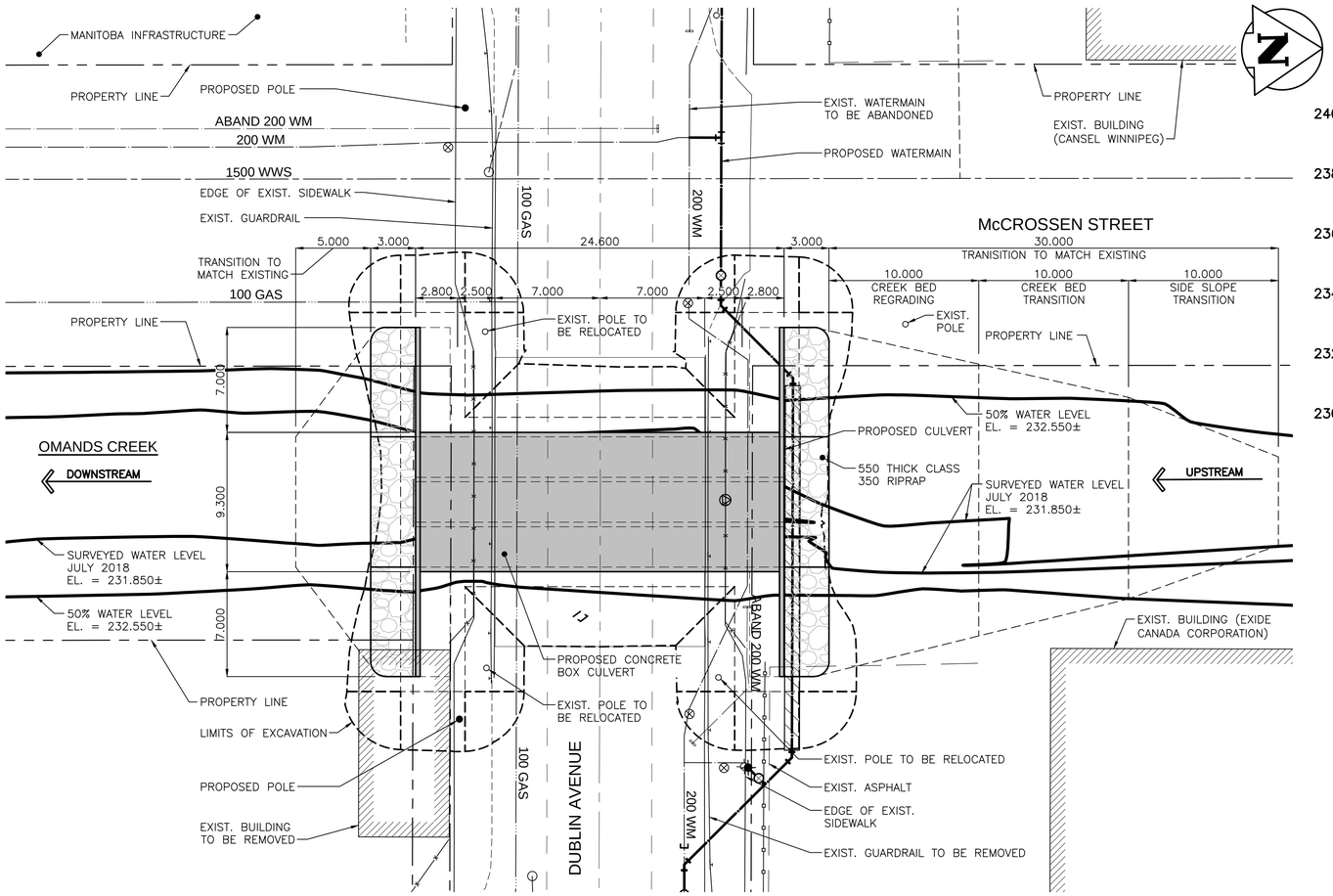
STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec will not be responsible to any party for damages incurred as a result of failing to notify Stantec that differing site or sub-surface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec, sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Stantec cannot be responsible for site work carried out without being present.

Appendix B
DRAWINGS



CULVERT PLAN
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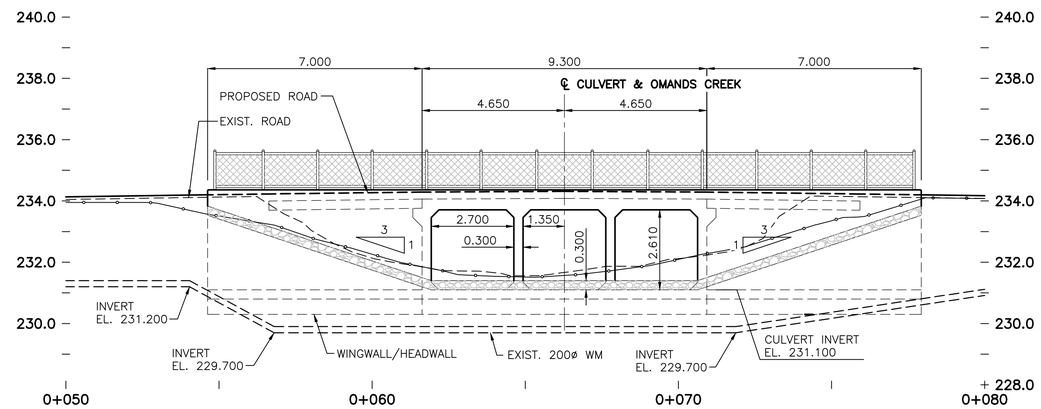
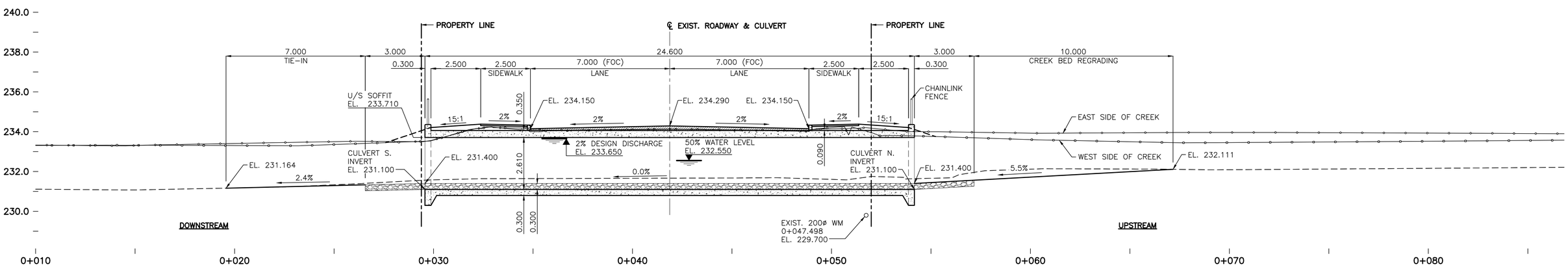


CULVERT ELEVATION – SOUTH END
1:100



CULVERT SECTION
1:100

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE — DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M. ELEV.

NO.	REVISIONS	DATE	BY
0	DRAFT PRELIMINARY DESIGN	18.12.03	M.J.B.

Stantec
500 - 311 Portage Avenue, Winnipeg MB Canada
www.stantec.com

DESIGNED BY	K.S.A.	CHECKED BY	K.S.A.
DRAWN BY	J.M.B.	APPROVED BY	
HOR. SCALE: VERTICAL:	AS NOTED	RELEASED FOR CONSTRUCTION:	
DATE	2018.12.03	DATE	

ENGINEER'S SEAL
PRELIMINARY DRAWING
NOT TO BE USED FOR CONSTRUCTION

CONSULTANT DRAWING NO.
S-101

Winnipeg
THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION

DUBLIN AVENUE BRIDGE REPLACEMENT
PROPOSED CULVERT STRUCTURE
GENERAL ARRANGEMENT

SHEET 4 OF 35
CAD FILE DRAWING NUMBER 33340s-101-cga.dwg
CITY DRAWING NUMBER P-3503-4

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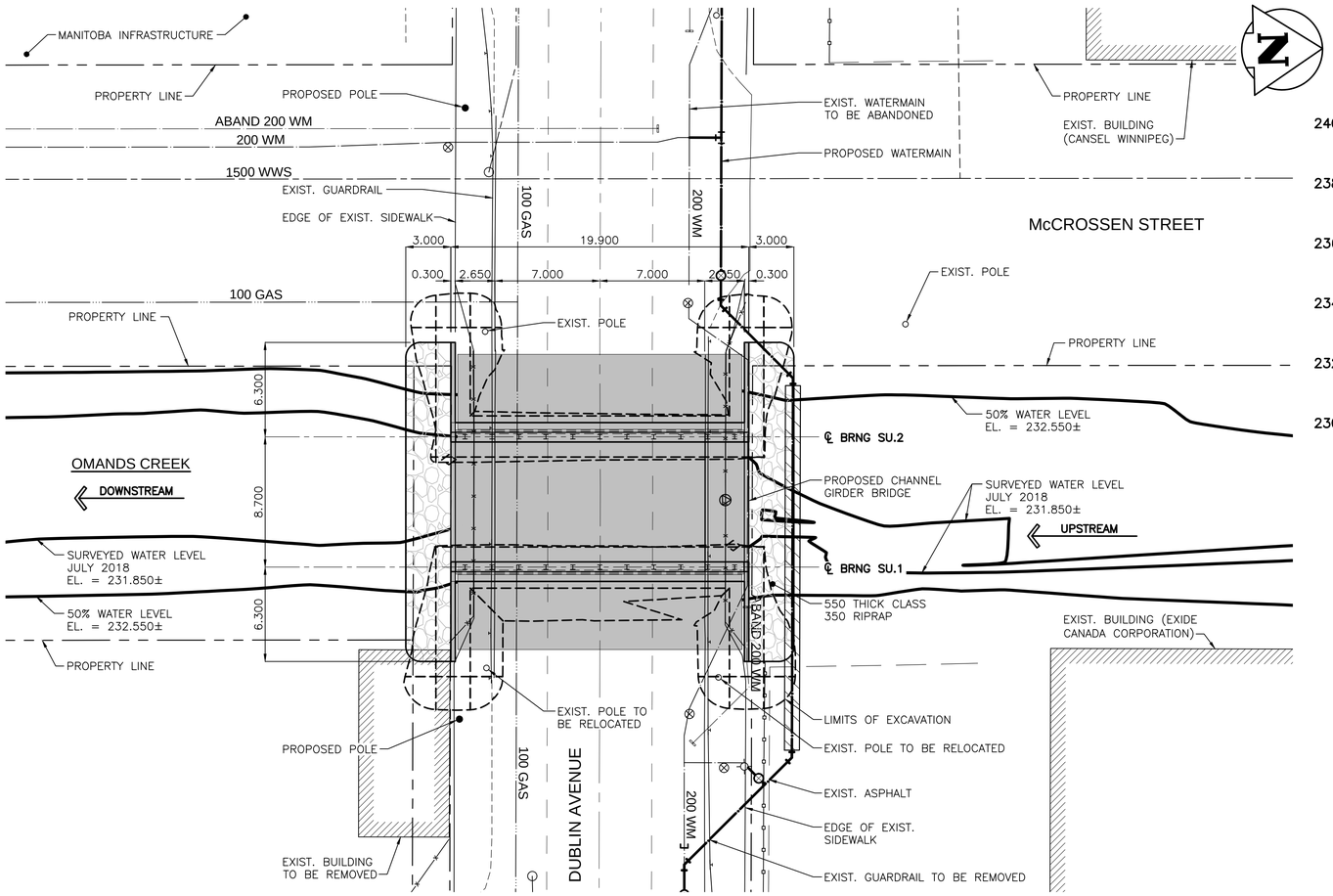
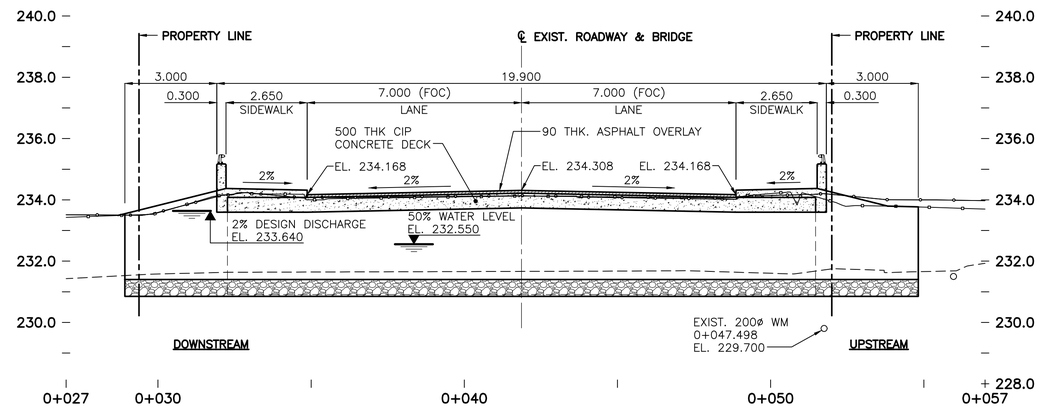
2018/12/03 9:44 AM By: Beernaerts, James



BRIDGE PLAN
1:200



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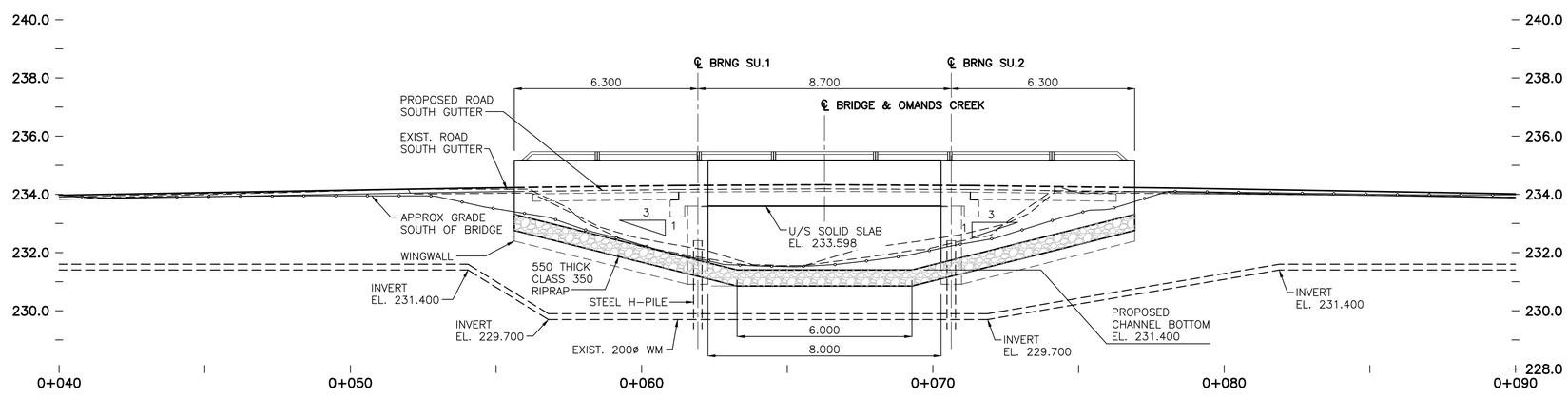

BRIDGE ELEVATION – SOUTH END
1:100

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE — DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M. ELEV.

NO.	REVISIONS	DATE	BY

Stantec
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DESIGNED BY	K.S.A.	CHECKED BY	K.S.A.
DRAWN BY	J.M.B.	APPROVED BY	
HOR. SCALE: VERTICAL:	AS NOTED	RELEASED FOR CONSTRUCTION:	
DATE	2018.12.03	DATE	

ENGINEER'S SEAL

CONSULTANT DRAWING NO.
S-201

Winnipeg
THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION

DUBLIN BRIDGE OVER OMANDS
PROPOSED BRIDGE STRUCTURE
GENERAL ARRANGEMENT

SHEET 6 OF 35
CAD FILE DRAWING NUMBER 33340s-201-bga.dwg
CITY DRAWING NUMBER P-3503-6

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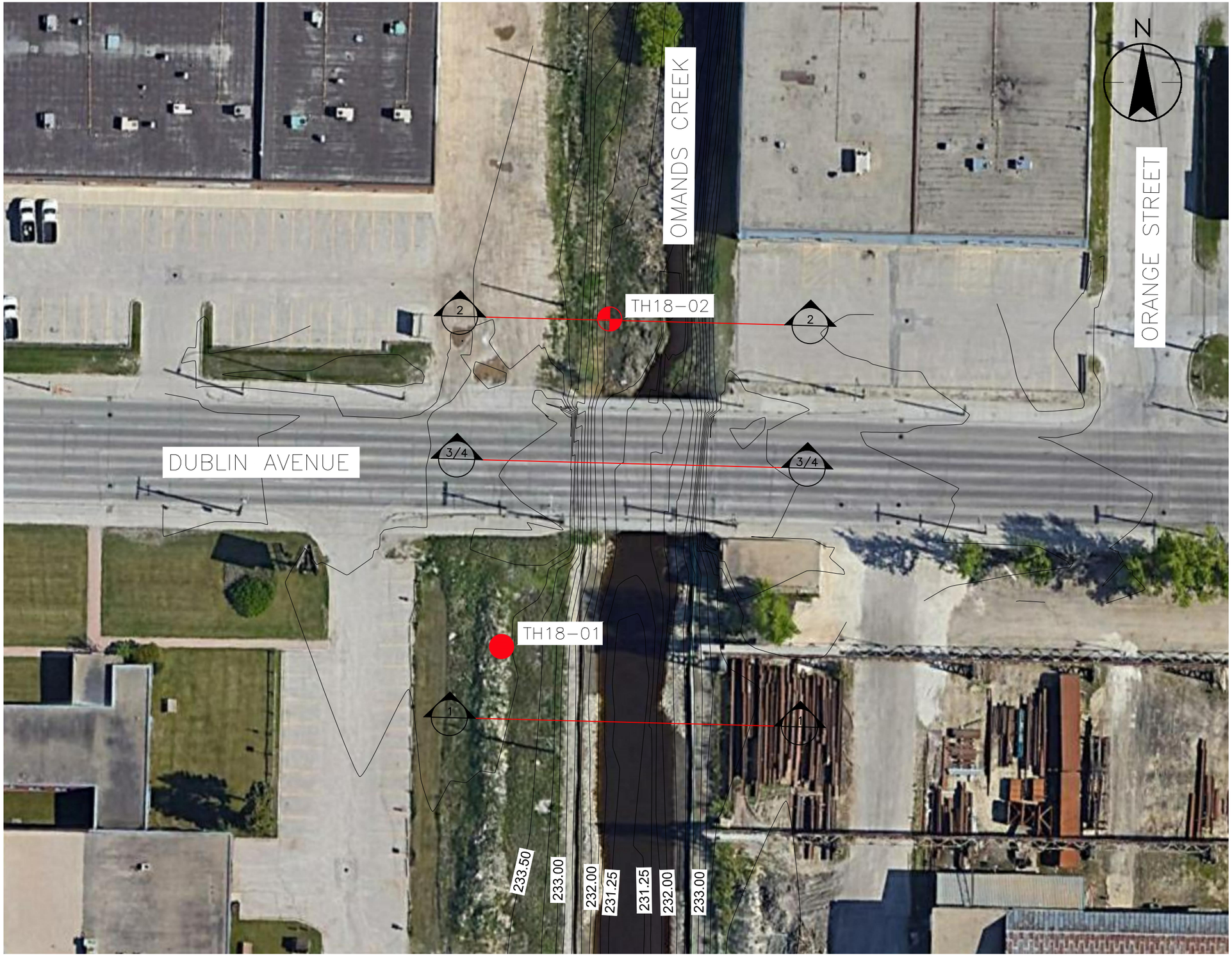

Stantec

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Winnipeg MB Canada R3B 2B9
Tel. 204.489.5900 Fax. 204.453.9012
www.stantec.com

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Legend

- TESTHOLE
- TESTHOLE WITH VIBRATING WIRE PIEZOMETER
- SURVEY CONTOUR
- SLOPE STABILITY CROSS SECTION

Notes

Revision	By	Appd.	YY.MM.DD
5			
4			
3			
2			
1			

Issued	By	Appd.	YY.MM.DD
E			
D			
C			
B			
A			

File Name: 33340s-1s-brdg1-lb

Dwn.	Chkd.	Dsgn.	YY.MM.DD

Permit-Seal

Client/Project

CITY OF WINNIPEG

REPLACEMENT OF THE DUBLIN AVENUE BRIDGE OVER OMAND'S CREEK

Winnipeg, Manitoba

Title

TESTHOLE LOCATION PLAN

Project No. 113733340

Scale 1:500 (0 5 15)

Drawing No. G-101

Sheet 1 of 1

Revision 0

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2018/11/26 4:21 PM By: Boughton, Lee

ORIGINAL SHEET - ISO 11x17 - v14.06

Appendix C
TESTHOLE RECORDS

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Rootmat	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor Quality*
25-50	*Poor Quality*
50-75	*Fair Quality*
75-90	*Good Quality*
90-100	*Excellent Quality*

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	*Crushed*
Severely Fractured	*Shattered or Very Blocky*
Fractured	*Blocky*
Moderately Jointed	*Sound*
Intact	*Very Sound*

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

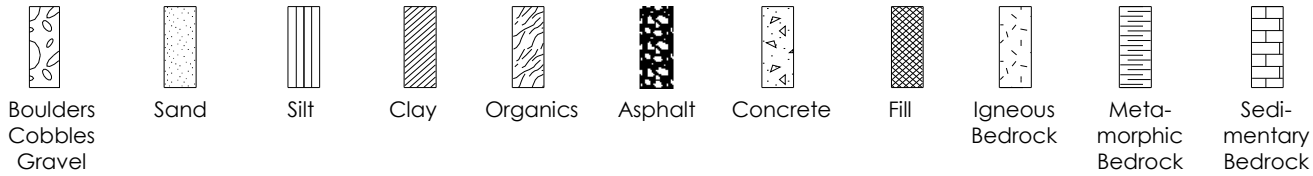
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

measured in standpipe, piezometer, or well

inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

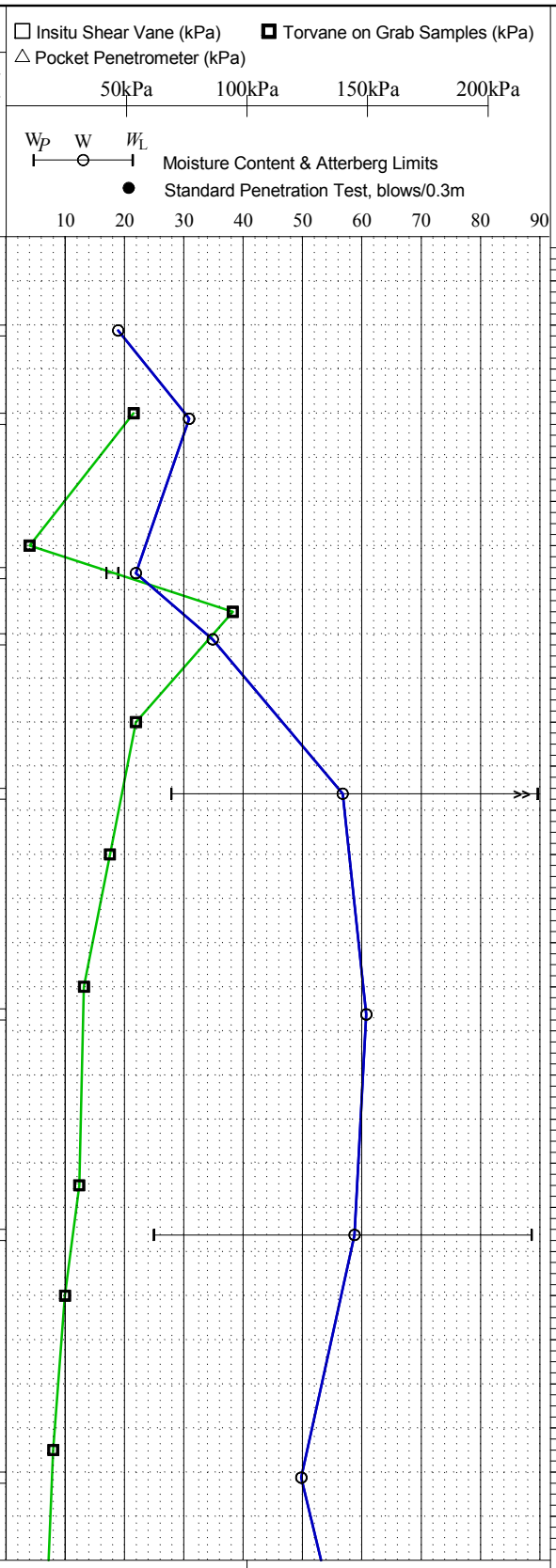
	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

TH18-01 TESTHOLE RECORD

CLIENT City of Winnipeg
PROJECT Dublin Avenue Bridge Replacement
LOCATION Dublin Avenue at Omand's Creek
DRILLING DATE August 3, 2018
DRILLING CO. Paddock Drilling Ltd.
DATUM Geodetic
ELEVATION 233.883 m
DRILLING METHOD 125 mm SSA
PROJECT No. 113733340
NORTHING 5530309.014
EASTING 629555.137

DEPTH (m)	SOIL TYPE	SOIL DESCRIPTION	SAMPLES TYPE	SAMPLES NUMBER	MOISTURE CONTENT (%)
0	TP	Brown to black TOPSOIL (TP) - with rootlets, some sand			
	FL	FILL: brown clay with sand, with gravel, with silt	GS		19
1	CH	Stiff black to grey fat CLAY (CH) - with to some silt	GS		31
2	ML	Very soft tan SILT (ML) - with fine grained sand, trace clay, trace gravel - trace gravel at 1.8 m	GS		22
		Particle Size Analysis Results from SPT @ 2.3 m: - 0% Gravel, 7% Sand, 79% Silt, 14% Clay	GS		35
3	CH	Stiff brown to grey fat CLAY (CH) - trace to some silt, trace sand - mottled brown and grey below 3.0 m - 2 mm silt seam at 3.7 m.	GS		57
4		Particle Size Analysis Results @ 3.8 m: - 0.2% Gravel, 0.5% Sand, 9.7% Silt, 89.6% Clay - firm below 4.4 m - trace silt inclusions at 4.6 m			
5			GS		61
6		- trace oxidation below 5.8 m - grey below 6.1 m			
7		Particle Size Analysis Results @ 6.9 m: - 0% Gravel, 0.9% Sand, 21.5% Silt, 77.6% Clay - soft below 7.0 m	GS		59
8					
9		- some silt inclusions at 8.4 m	GS		50

WELL DATA



DEPTH (ft): 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30

Sample Type: GS - Grab Sample SS - Split Spoon RC - Rock Core ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite Drill Cuttings Sand Slough

Logged by: Ivanna Montani
Reviewed by: Rhett Bonham

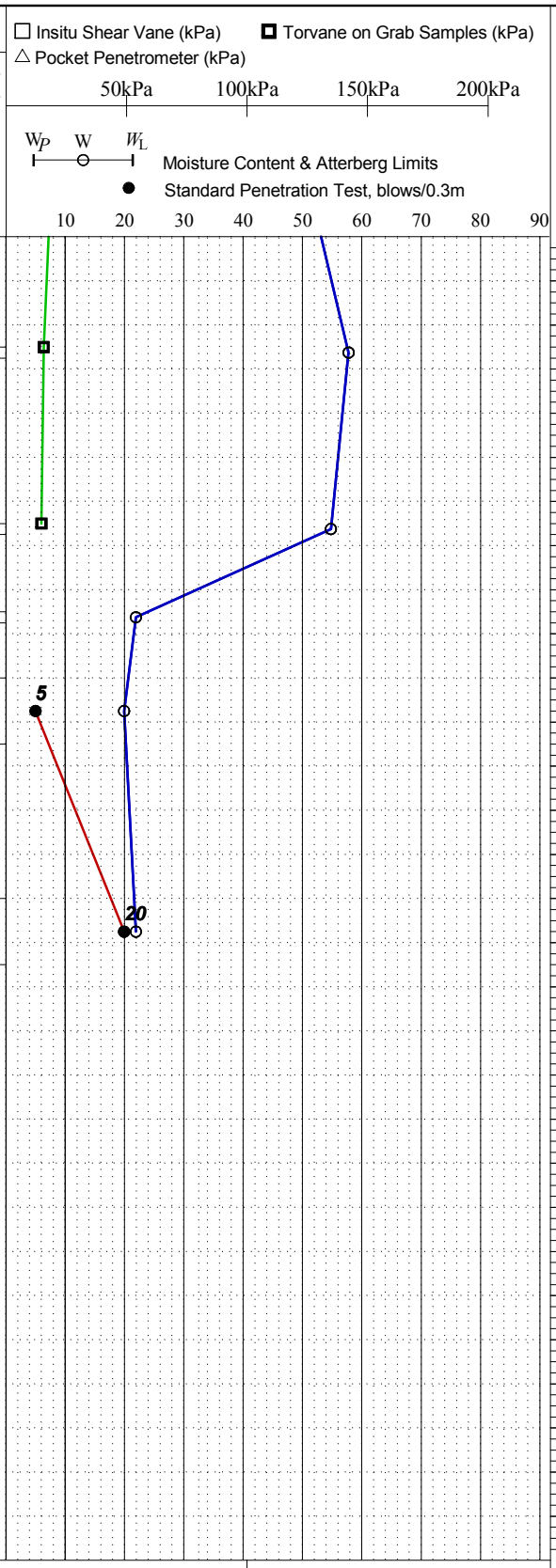
Stantec

TH18-01 TESTHOLE RECORD

cont'd

CLIENT City of Winnipeg
PROJECT Dublin Avenue Bridge Replacement
LOCATION Dublin Avenue at Omand's Creek
DRILLING DATE August 3, 2018
DRILLING CO. Paddock Drilling Ltd.
DATUM Geodetic
ELEVATION 233.883 m
DRILLING METHOD 125 mm SSA
PROJECT No. 113733340
NORTHING 5530309.014
EASTING 629555.137

DEPTH (m)	SOIL TYPE	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTURE CONTENT (%)
10		- trace gravel at 9.9 m	GS		58
11			GS		55
12	SM	Loose grey SILTY SAND (SM) - fine grained sand	GS		22
13	SW	Loose tan to brown SAND (SW) - medium to coarse grained sand, occasional cobbles, trace large gravel	SS		20
14	SW	- compact below 13.7 m - no sample recovery below 14.0 m	SS		22
15	TL	Inferred silt TILL (TL) - frequent cobbles encountered during drilling below 14.6 m			
16		End of Testhole • Auger refusal at a depth of 15.4 m on suspected dense till. • Moderate groundwater seepage was observed at a depth of 11.6 m. • Groundwater level was observed at a depth of 8.5 m upon completion of drilling. • Soil sloughing was observed in the sand layer at a depth of 12.2 m. • Testhole backfilled with auger cuttings and bentonite chips. • Vibrating wire piezometer ID 100D1800939 installed with tip at Elev. 224.97 m immediately adjacent to testhole TH18-01.			
17					
18					



DEPTH (ft): 30, 32, 34, 36, 38, 40, 42, 44, 46, 48, 50, 52, 54, 56, 58, 60

Sample Type: GS - Grab Sample SS - Split Spoon RC - Rock Core ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite Drill Cuttings Sand Slough

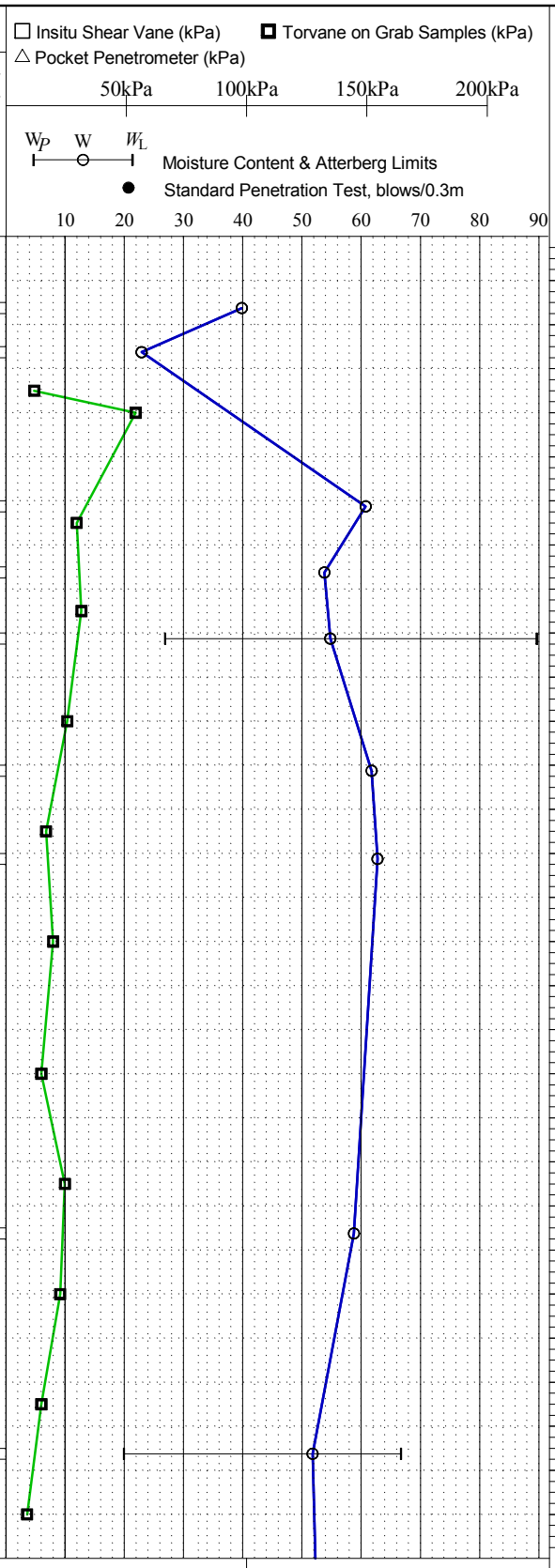
Logged by: Ivanna Montani
Reviewed by: Rhett Bonham

Stantec

TH18-02 TESTHOLE RECORD

CLIENT City of Winnipeg
PROJECT Dublin Avenue Bridge Replacement
LOCATION Dublin Avenue at Omand's Creek
DRILLING DATE August 3, 2018
DRILLING CO. Paddock Drilling Ltd.
DATUM Geodetic
ELEVATION 232.105 m
DRILLING METHOD 125 mm SSA
PROJECT No. 113733340
NORTHING 5530351.932
EASTING 629569.771

DEPTH (m)	SOIL TYPE	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTURE CONTENT (%)
0	TP	Black TOPSOIL (TP) - with rootlets, some sand			
	CH	Firm grey fat CLAY (CH) - with to some silt, with to some sand, trace rootlets	GS		40
			GS		23
1	ML	Soft tan SILT (ML) - some fine grained sand, trace clay, trace gravel			
		Stiff brown to grey fat CLAY (CH) - trace to some silt, trace silt inclusions, trace sand	GS		61
2		- firm, mottled grey to brown below 1.5 m	GS		54
		- trace oxidation below 2.3 m			
		Particle Size Analysis Results @ 2.7 m: - 0% Gravel, 0.9% Sand, 10.1% Silt, 89.0% Clay	GS		55
3		- soft, grey below 3.4 m			
			GS		62
4			GS		63
5					
6	CH	- soft to firm at 6.6 m			
7			GS		59
		- trace gravel, increasing sand content below 7.6 m			
8		Particle Size Analysis Results @ 8.4 m: - 0.2% Gravel, 8.2% Sand, 27.5% Silt, 64.1% Clay	GS		52
9					



DEPTH (ft): 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30

Sample Type: GS - Grab Sample, SS - Split Spoon, RC - Rock Core, ST - Shelby Tube, PT - Piston Tube, VT - Shear Vane Test

Piezometer Backfill Type: Bentonite, Drill Cuttings, Sand, Slough

Logged by: Ivanna Montani
Reviewed by: Rhett Bonham

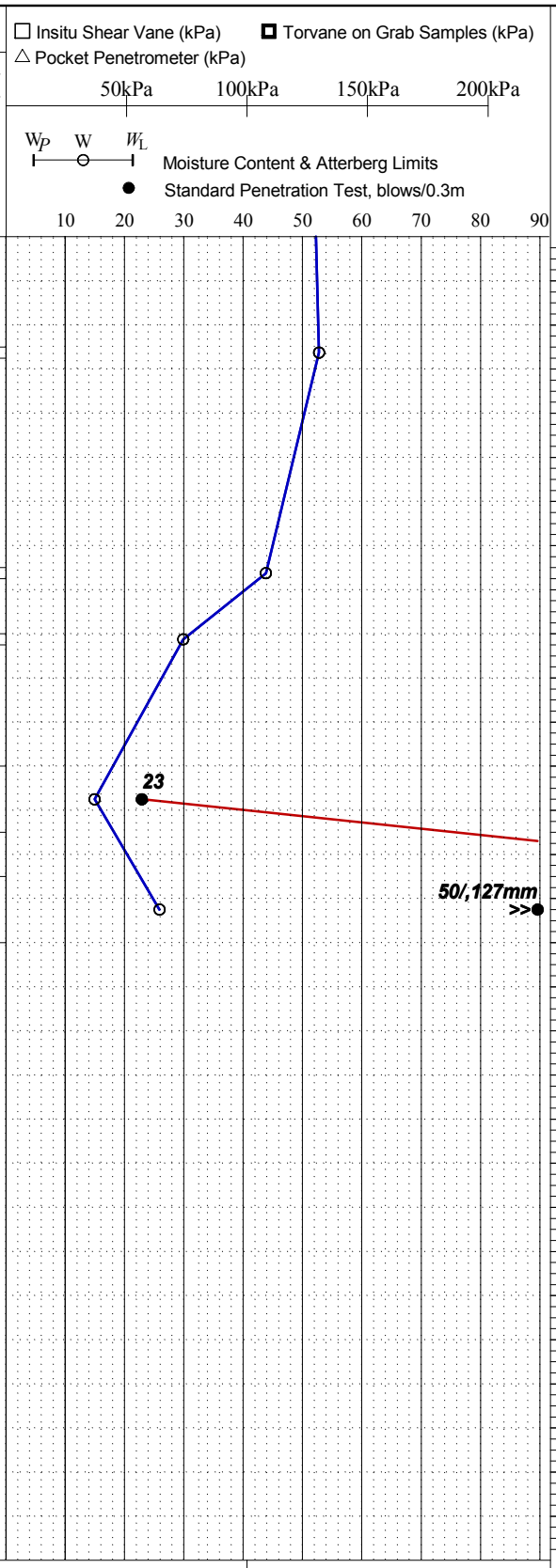
Stantec

TH18-02 TESTHOLE RECORD

cont'd

CLIENT City of Winnipeg
PROJECT No. 113733340
PROJECT Dublin Avenue Bridge Replacement
DATUM Geodetic
NORTHING 5530351.932
LOCATION Dublin Avenue at Omand's Creek
ELEVATION 232.105 m
EASTING 629569.771
DRILLING DATE August 3, 2018
DRILLING CO. Paddock Drilling Ltd.
DRILLING METHOD 125 mm SSA

DEPTH (m)	SOIL TYPE	SOIL SYMBOL	SOIL DESCRIPTION	WELL DATA	SAMPLES TYPE	SAMPLES NUMBER	MOISTURE CONTENT (%)
			- very soft below 8.8 m - increasing silt below 9.1 m				
10					GS		53
			- intermixed silt till with clay below 10.7 m				
11							
			- frequent silt inclusions below 11.3 m		GS		44
12					GS		30
			Loose grey SILT TILL (TL) - with to some clay, with sand, some gravel				
13	TL		- compact below 12.8 m		SS		15
			- very dense below 13.4 m		SS		26
14							
15			End of Testhole • Auger refusal at a depth of 14.0 m on suspected dense till. • Moderate to heavy groundwater seepage was observed at a depth of 12.2 m. • Groundwater level was observed at a depth of 8.2 m. • No soil sloughing was observed during or upon completion of drilling. • Testhole backfilled with auger cuttings and bentonite chips.				
16							
17							
18							



DEPTH (ft): 30, 32, 34, 36, 38, 40, 42, 44, 46, 48, 50, 52, 54, 56, 58, 60

Sample Type: GS - Grab Sample SS - Split Spoon RC - Rock Core ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite Drill Cuttings Sand Slough

Logged by: Ivanna Montani

Reviewed by: Rhett Bonham

Stantec

Appendix D
SITE PHOTOGRAPHS

Photo 1: South of bridge (downstream), looking south at previously remediated area

Photo 2: South of bridge (downstream), looking northwest at small diameter rockfill

Photo 3: North of bridge (upstream), looking northeast at steep east bank

Photo 4: North of bridge (upstream), looking south at east bank

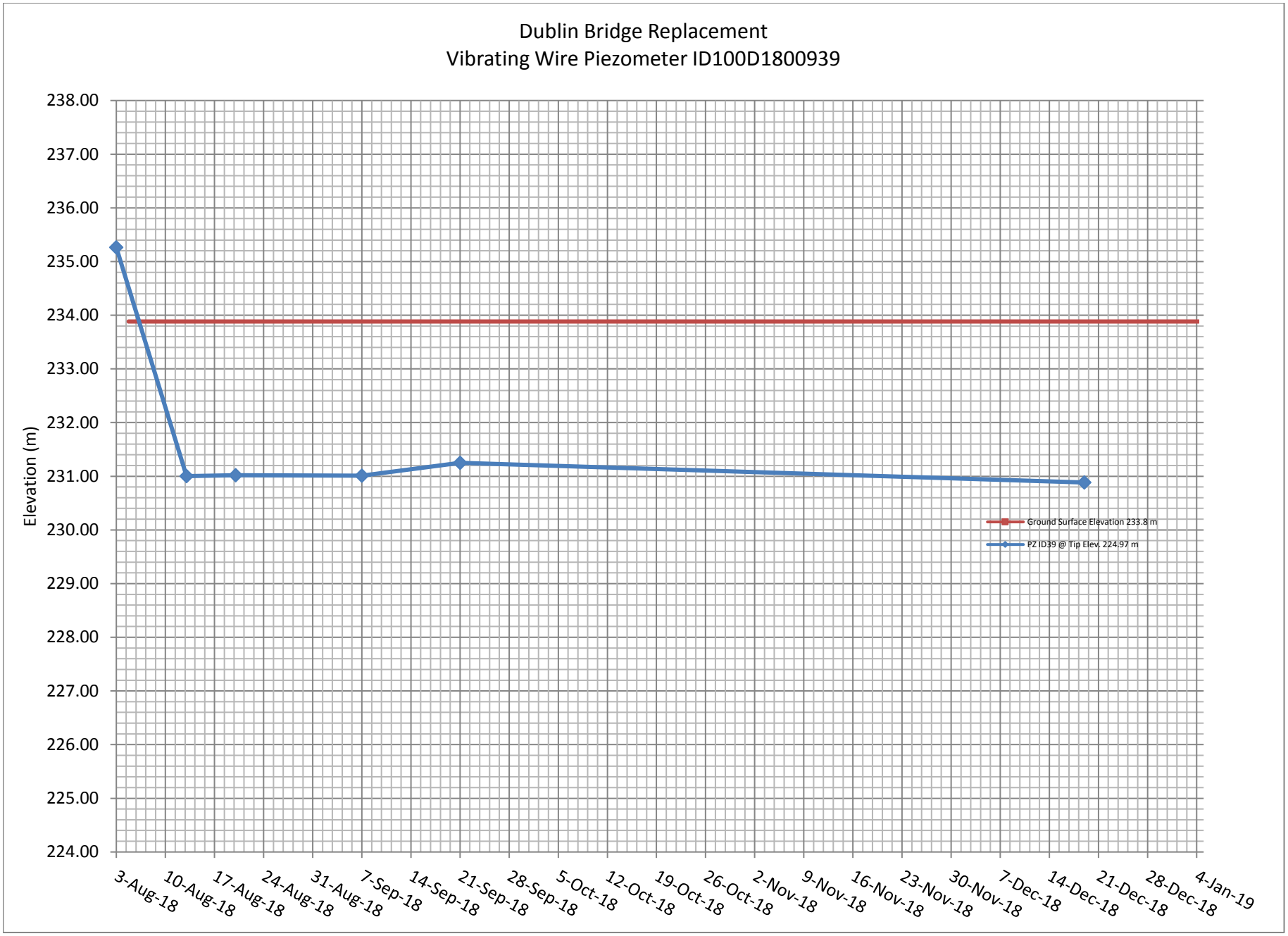
Photo 5: North of bridge (upstream), looking west

Photo 6: North of bridge (upstream), looking west

Photo 7: North of bridge (upstream), looking north along flat lower bank area, drill cuttings in the foreground

Photo 8: North of bridge (upstream), looking north at typical creek bottom and vegetation

Appendix E
PIEZOMETER DATA


Dublin Bridge Replacement
Vibrating Wire Piezometer ID100D1800939
Elevation (m)
238.00
237.00
236.00
235.00
234.00
233.00
232.00
231.00
230.00
229.00
228.00
227.00
226.00
225.00
224.00
3-Aug-18
10-Aug-18
17-Aug-18
24-Aug-18
31-Aug-18
7-Sep-18
14-Sep-18
21-Sep-18
28-Sep-18
5-Oct-18
12-Oct-18
19-Oct-18
26-Oct-18
2-Nov-18
9-Nov-18
16-Nov-18
23-Nov-18
30-Nov-18
7-Dec-18
14-Dec-18
21-Dec-18
28-Dec-18
4-Jan-19
Ground Surface Elevation 233.8 m
PZ ID39 @ Tip Elev. 224.97 m

Appendix F
LABORATORY TEST DATA

LABORATORY
199 Henlow Bay
Winnipeg MB R3Y 1G4
Tel: (204) 488-6999

PARTICLE SIZE ANALYSIS

ASTM D422

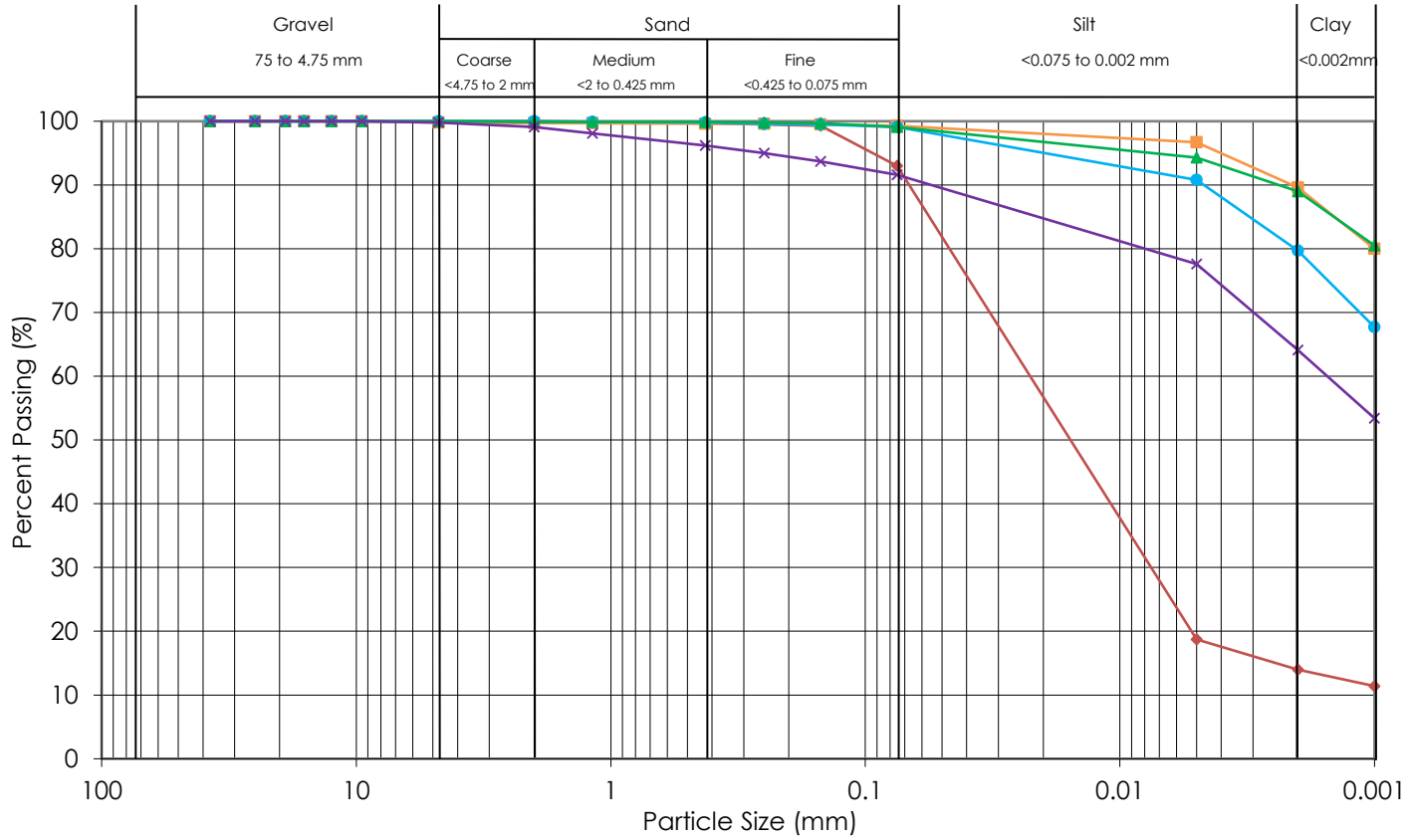
Stantec Consulting Ltd.
500-311 Portage Avenue
Winnipeg, Manitoba
R3B 2B9

Project No.: 113733340
Project Name: Dublin Bridge Replacement

Date Samples Received: August 17, 2018

Material Type: Fat Clay / Silt

Tested By: Nestor Abarca, C.Tech.



Symbol	Sample ID	Gravel, % 75 to 4.75 mm	Sand, % Coarse <4.75 to 2.0 mm	Sand, % Medium <2.0 to 0.425 mm	Sand, % Fine <0.425 to 0.075 mm	Silt, % <0.075 to 0.002 mm	Clay, % <0.002 mm	Colloids, % < 0.001 mm
◆	TH18-01 @ 2.3 m	0.0	0.0	0.3	6.7	79.0	14.0	11.4
■	TH18-01 @ 3.8 m	0.2	0.1	0.1	0.3	9.7	89.6	80.0
●	TH18-01 @ 6.9 m	0.0	0.0	0.2	0.7	19.4	79.7	67.7
▲	TH18-02 @ 2.7 m	0.0	0.1	0.0	0.8	10.1	89.0	80.5
×	TH18-02 @ 8.4 m	0.2	0.7	2.9	4.6	27.5	64.1	53.4

NT*: Sample not tested for colloids.

Reviewed By: Rhett Bonham, P.Eng.
Date Reviewed: September 4, 2018

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request. The data presented above is for the sole use of the client stipulated above. Stantec is not responsible, nor can be held liable, for the use of this report by any other party, with or without the knowledge of Stantec.

LABORATORY
199 Henlow Bay
Winnipeg MB R3Y 1G4
Tel: (204) 488-6999

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D4318

Stantec Consulting Ltd.
500-311 Portage Avenue
Winnipeg, Manitoba
R3B 2B9

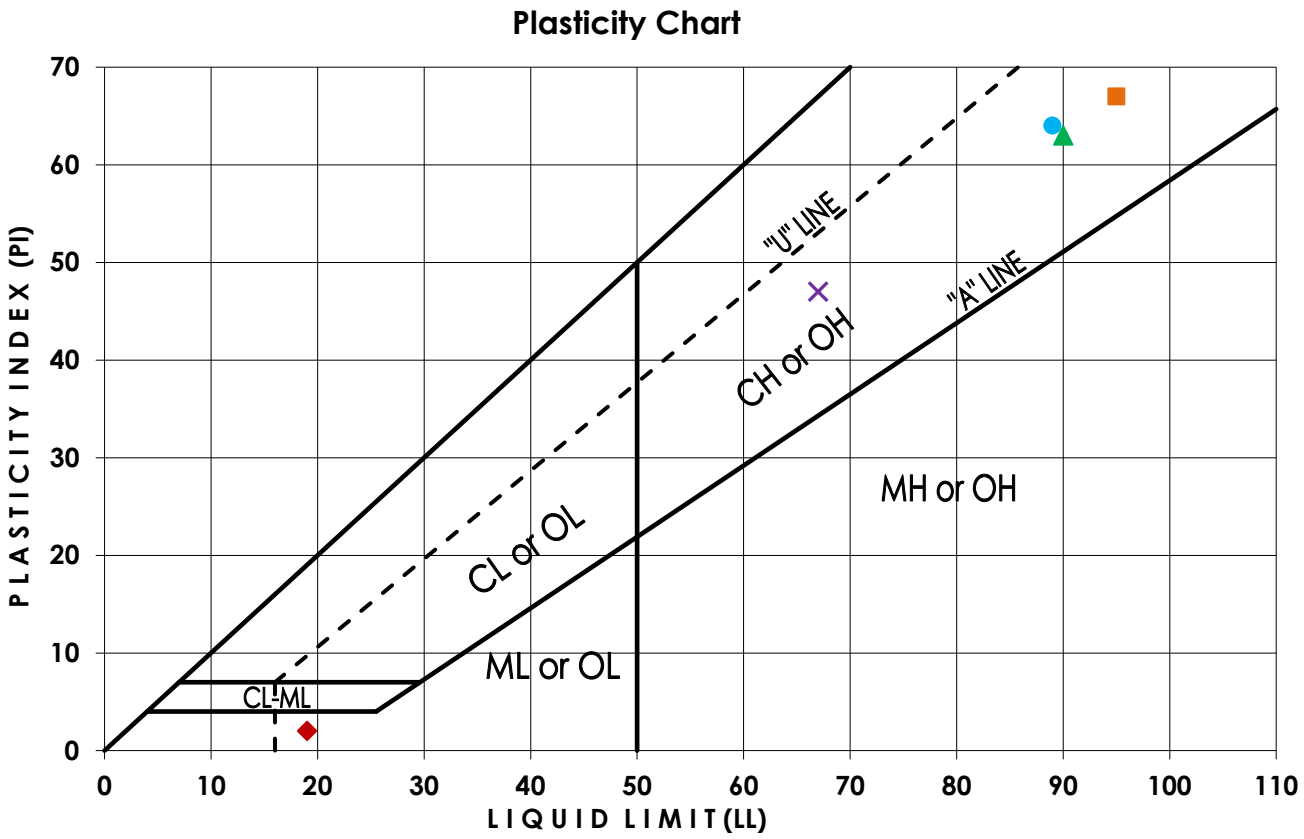
Project No.: 113733340
Project Name: Dublin Bridge Replacement

Date Samples Received: August 17, 2018

Material Type: Fat Clay / Silt

Tested By: Nestor Abarca, C. Tech.

Symbol	Testhole No.	Depth (m)	Liquid Limit	Plastic Limit	Plasticity Index	USCS
◆	TH18-01	2.3 m	19	17	2	ML
■	TH18-01	3.8 m	95	28	67	CH
●	TH18-01	6.9 m	89	25	64	CH
▲	TH18-02	2.7 m	90	27	63	CH
x	TH18-02	8.4 m	67	20	47	CH



Reviewed By: Rhett Bonham, P.Eng.
Date Reviewed: September 4, 2018

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request. The data presented above is for the sole use of the client stipulated above. Stantec is not responsible, nor can be held liable, for the use of this report by any other party, with or without the knowledge of Stantec.

Appendix G
SLOPE STABILITY OUTPUTS

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

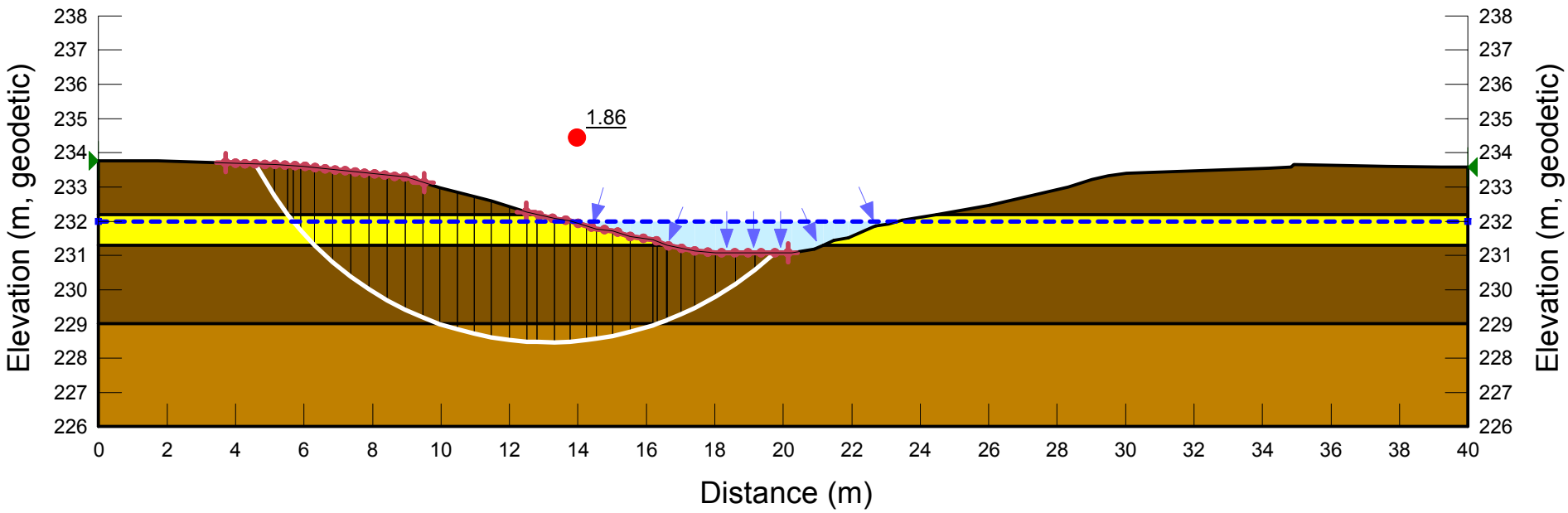


Figure 1 - Existing Conditions Approx. 25 m south of Bridge - West Bank - Normal GWL | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_creek_25 m_south_20190327.gsz

Stantec

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

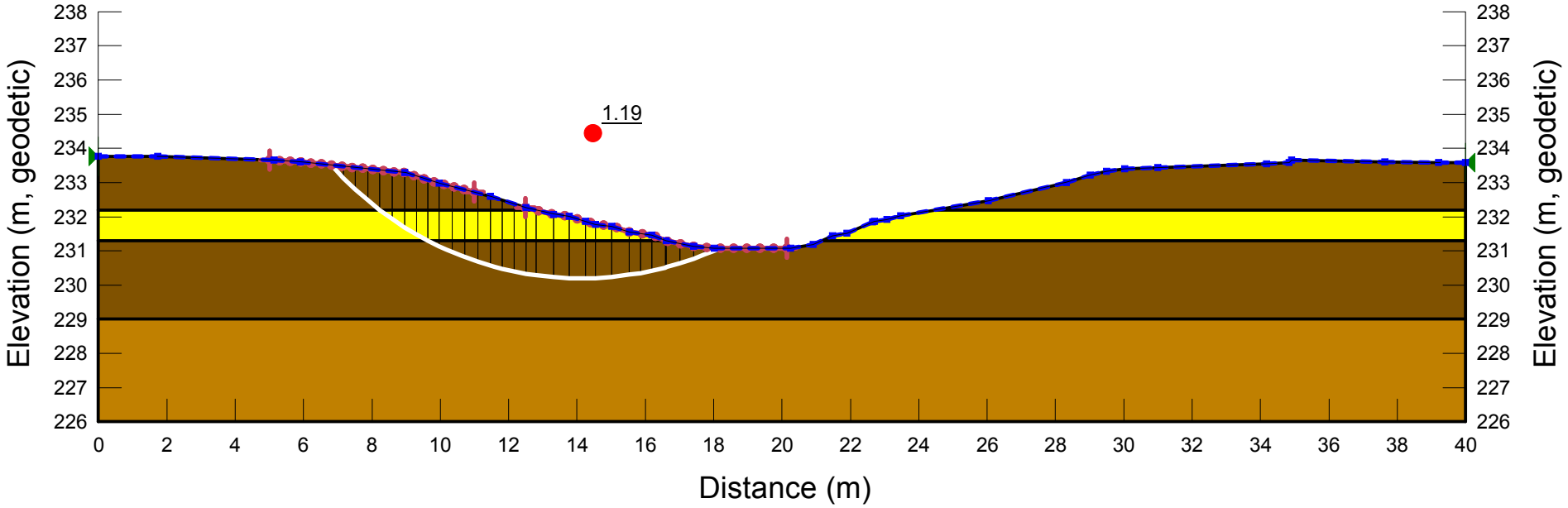


Figure 2 - Existing Conditions Approx. 25 m south of Bridge - West Bank - Critical GWL | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Stantec

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_creek_25 m_south_20190327.gsz

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

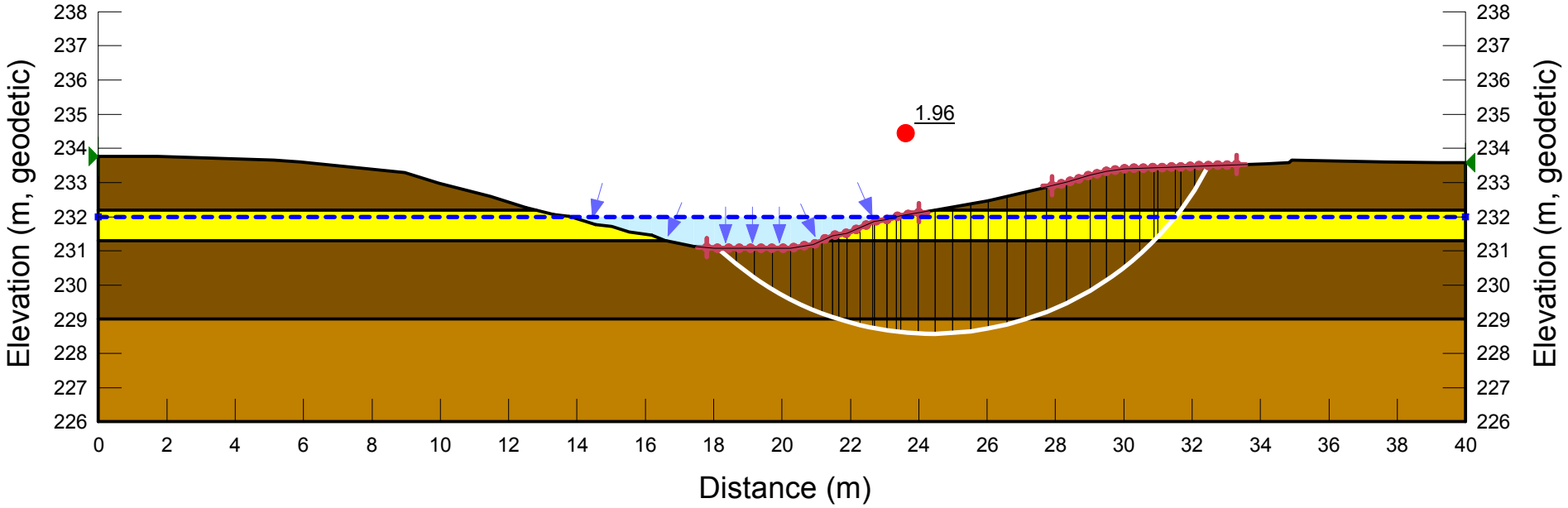


Figure 3 - Existing Conditions Approx. 25 m south of Bridge - East Bank - Normal GWL | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Stantec

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_creek_25 m_south_20190327.gsz

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

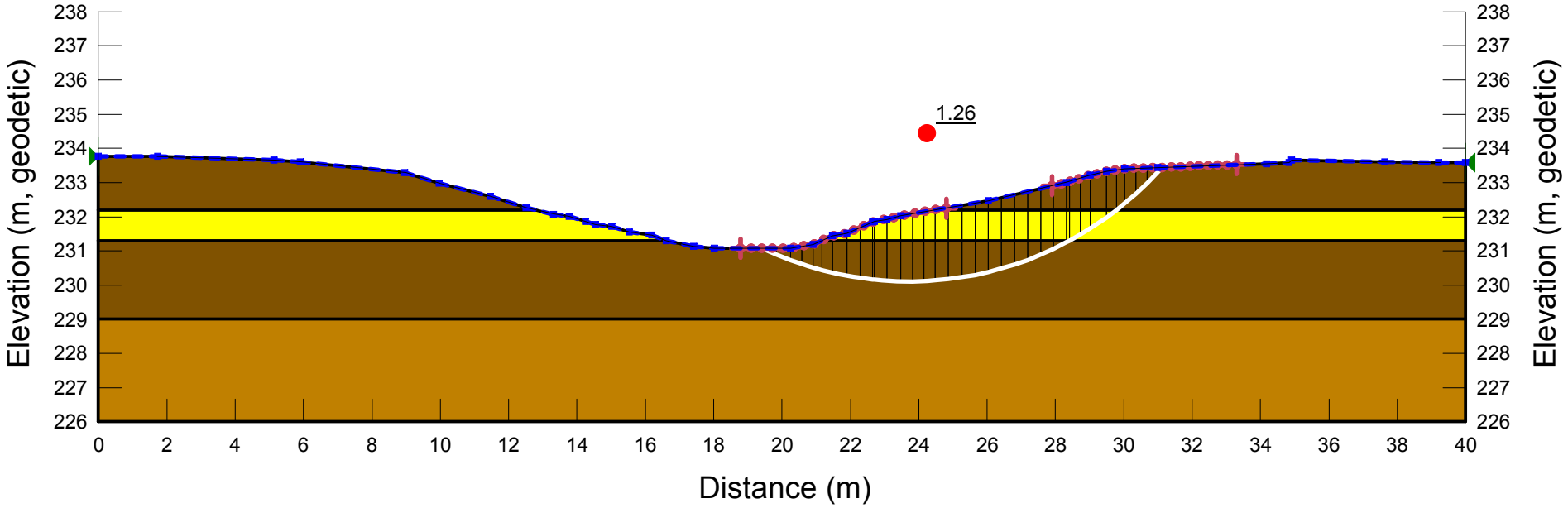


Figure 4 - Existing Conditions Approx. 25 m south of Bridge - East Bank - Critical GWL | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Stantec

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_creek_25 m_south_20190327.gsz

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

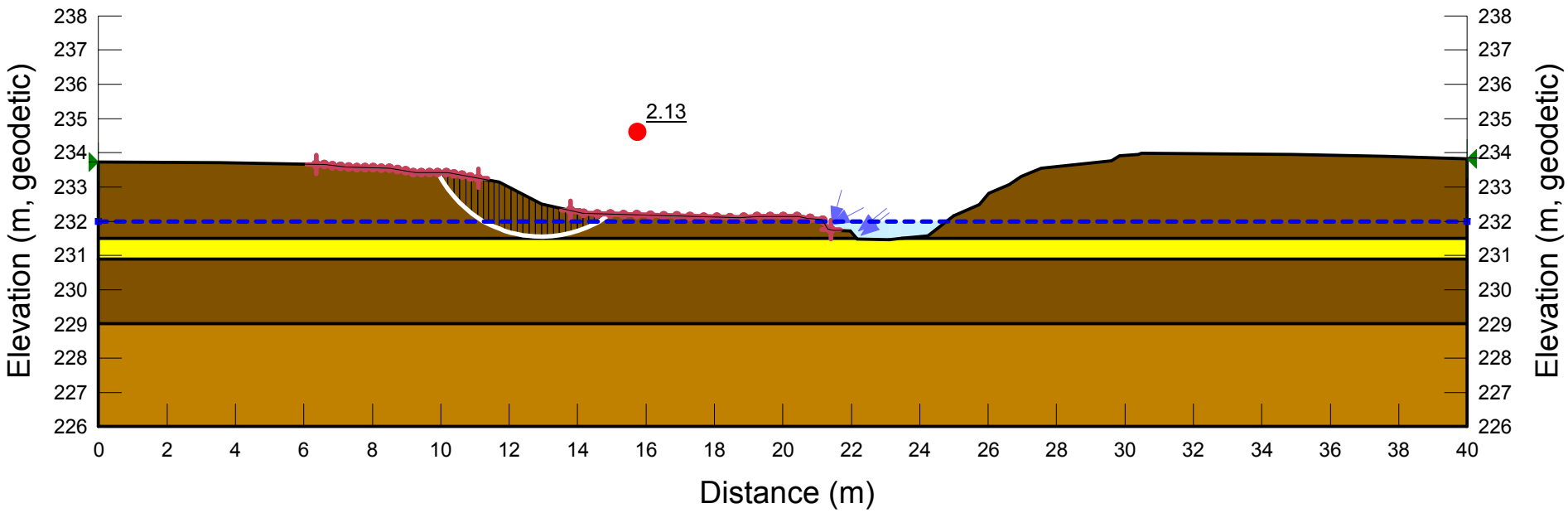


Figure 5 - Existing Conditions Approx. 10 m north of Bridge - West Bank - Normal GWL | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_creek_10 m_north_20190327.gsz

Stantec

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

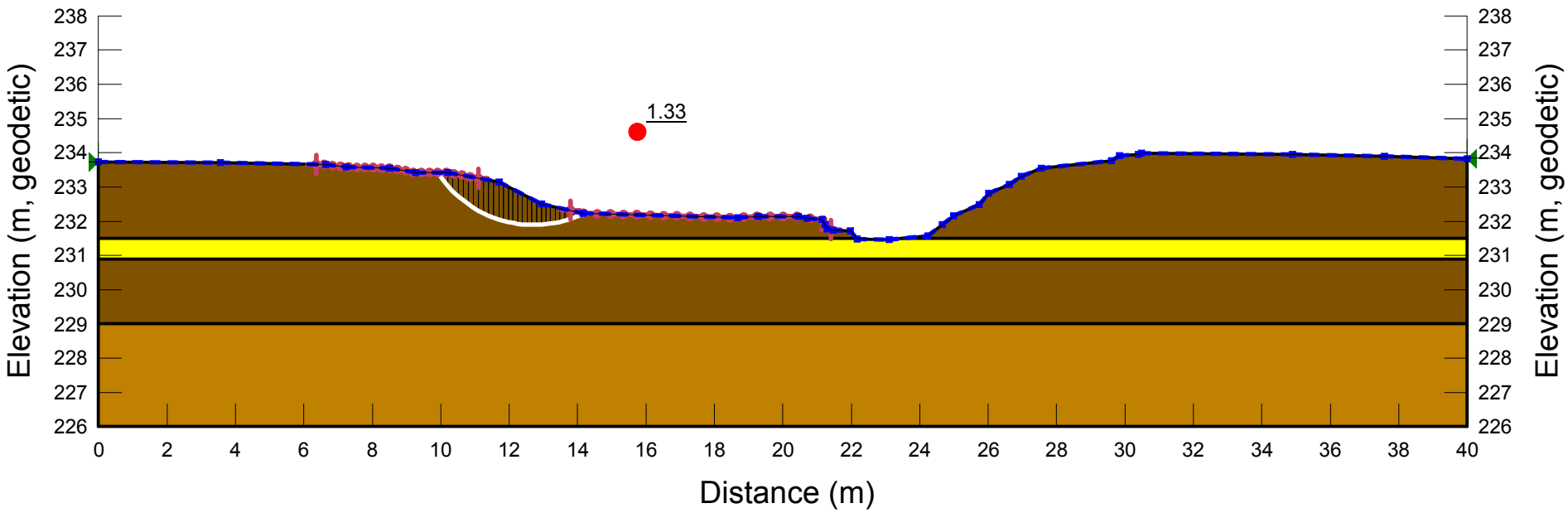


Figure 6 - Existing Conditions Approx. 10 m north of Bridge - West Bank - Critical GWL | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_creek_10 m_north_20190327.gsz

Stantec

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

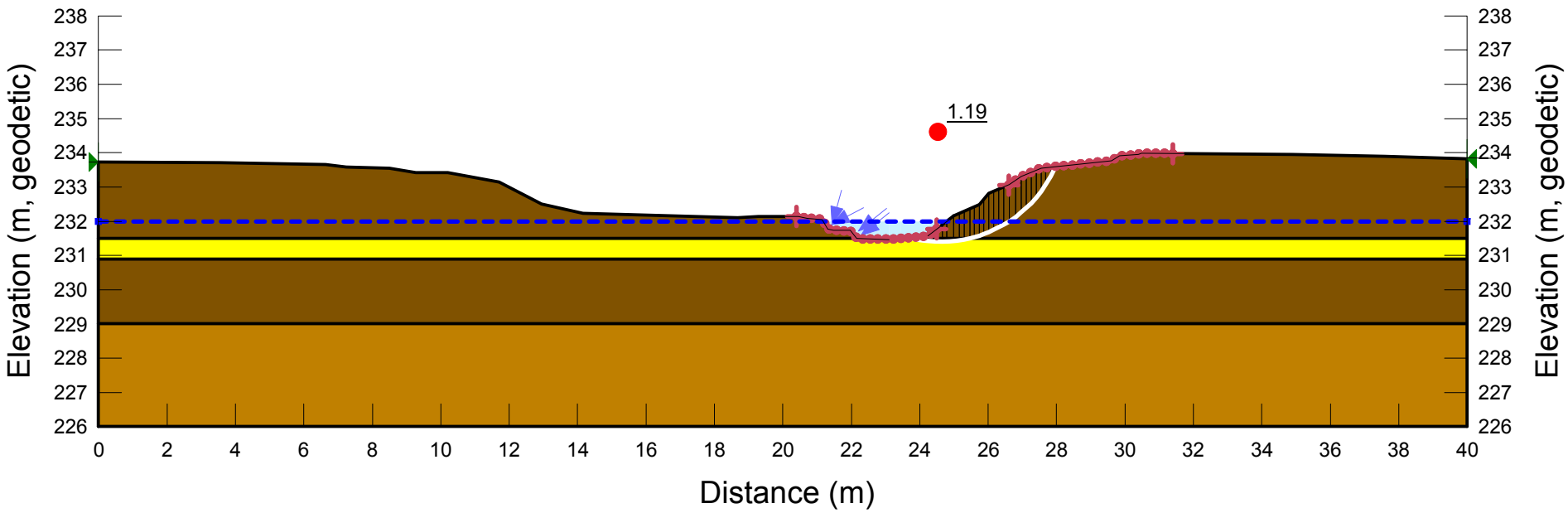


Figure 7 - Existing Conditions Approx. 10 m north of Bridge - East Bank - Normal GWL | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_creek_10 m_north_20190327.gsz

Stantec

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

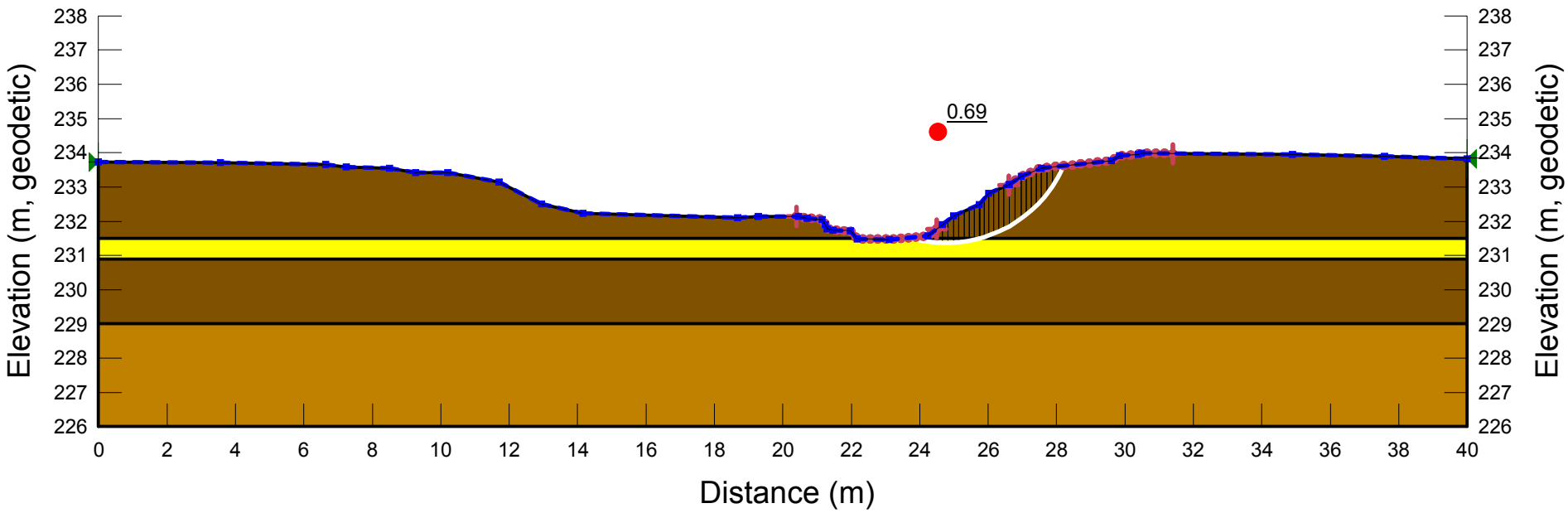


Figure 8 - Existing Conditions Approx. 10 m north of Bridge - East Bank - Critical GWL | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_creek_10 m_north_20190327.gsz

Stantec

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Rockfill Riprap	Mohr-Coulomb	20	0	35
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

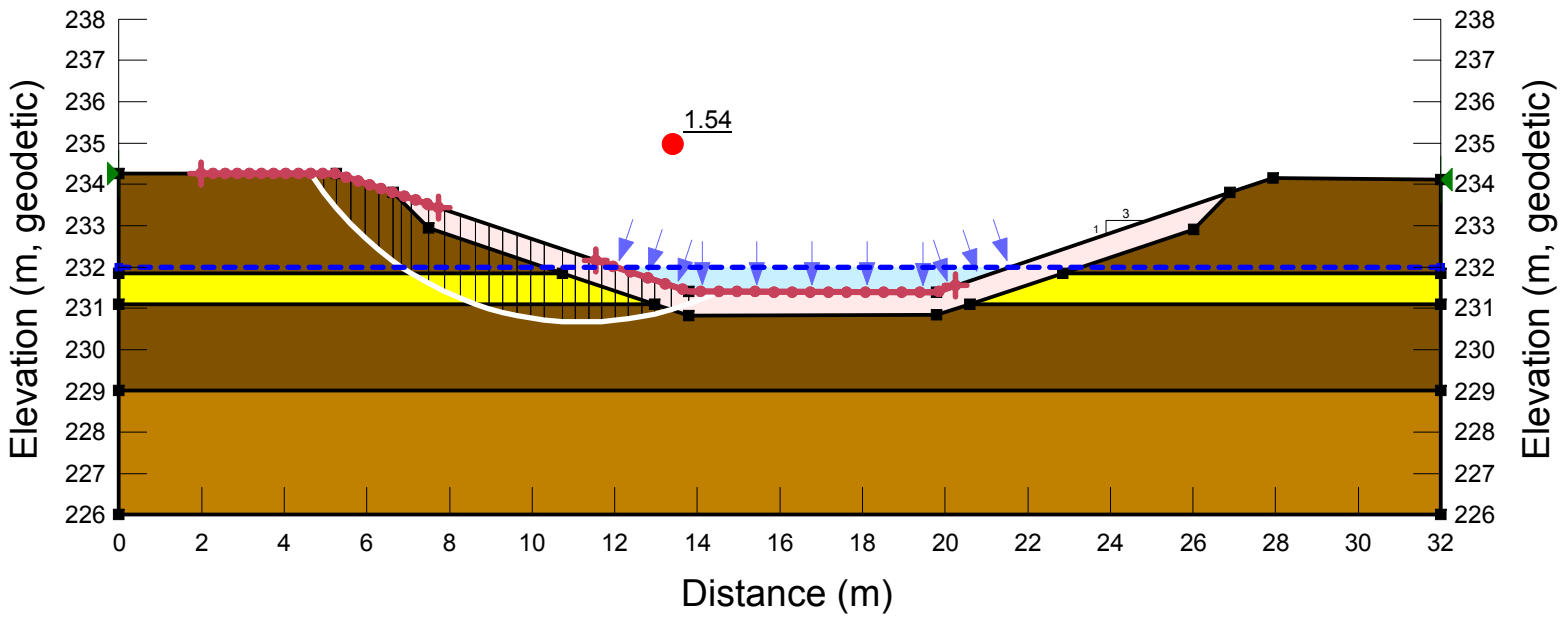


Figure 9 - Bridge Option - Normal Conditions | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_3H1V_bridge_20190327.gsz

Stantec

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Rockfill Riprap	Mohr-Coulomb	20	0	35
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

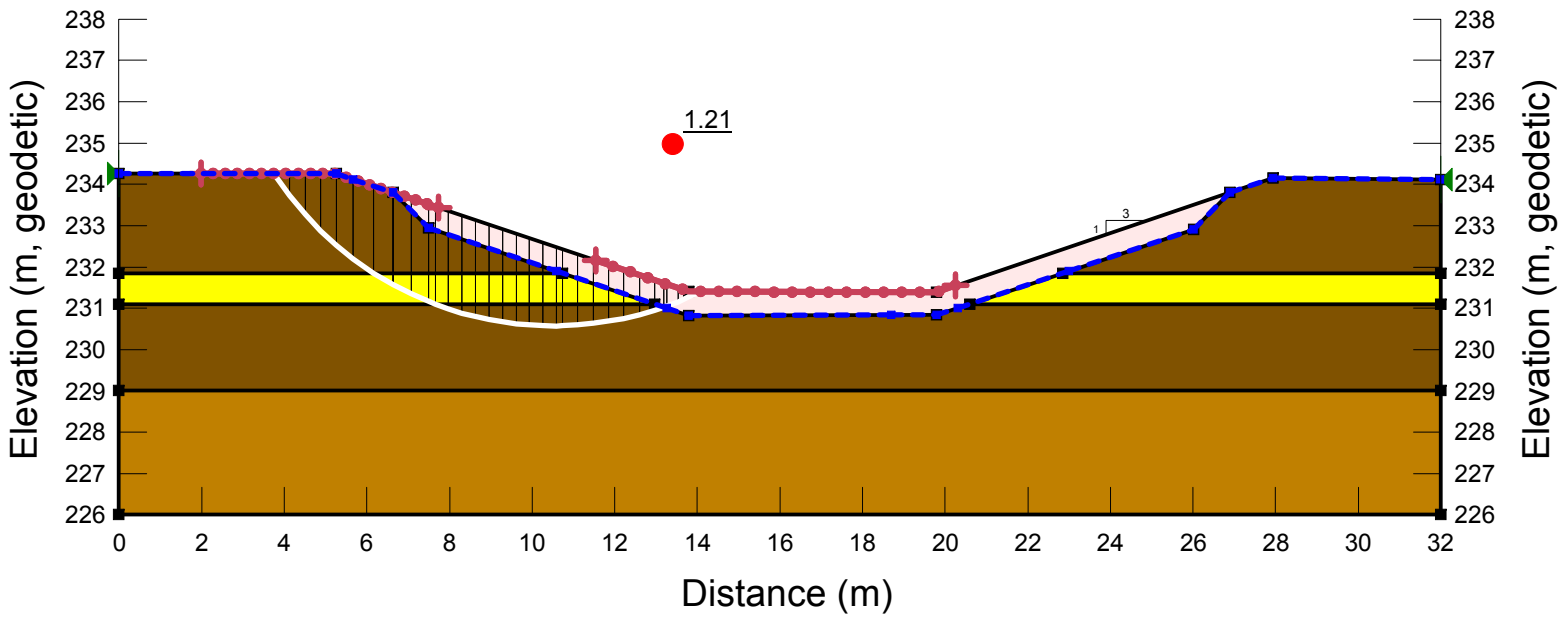


Figure 10 - Bridge Option - Critical Conditions | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_3H1V_bridge_20190327.gsz

Stantec

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Rockfill Riprap	Mohr-Coulomb	20	0	35
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

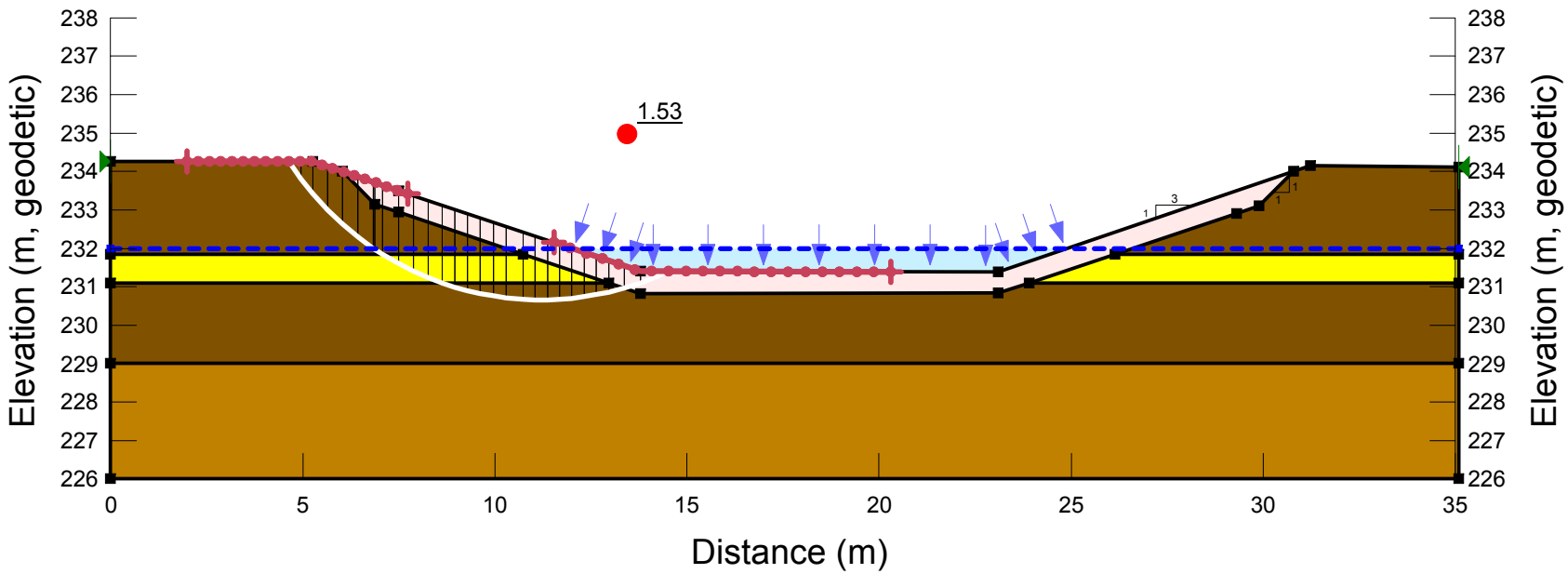


Figure 11 - Culvert Option - Normal Conditions | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_3H1V_culvert_20190327.gsz

Stantec

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Firm Clay	Mohr-Coulomb	17	2	20
	Rockfill Riprap	Mohr-Coulomb	20	0	35
	Silt	Mohr-Coulomb	18	1	25
	Soft Clay	Mohr-Coulomb	17	2	16

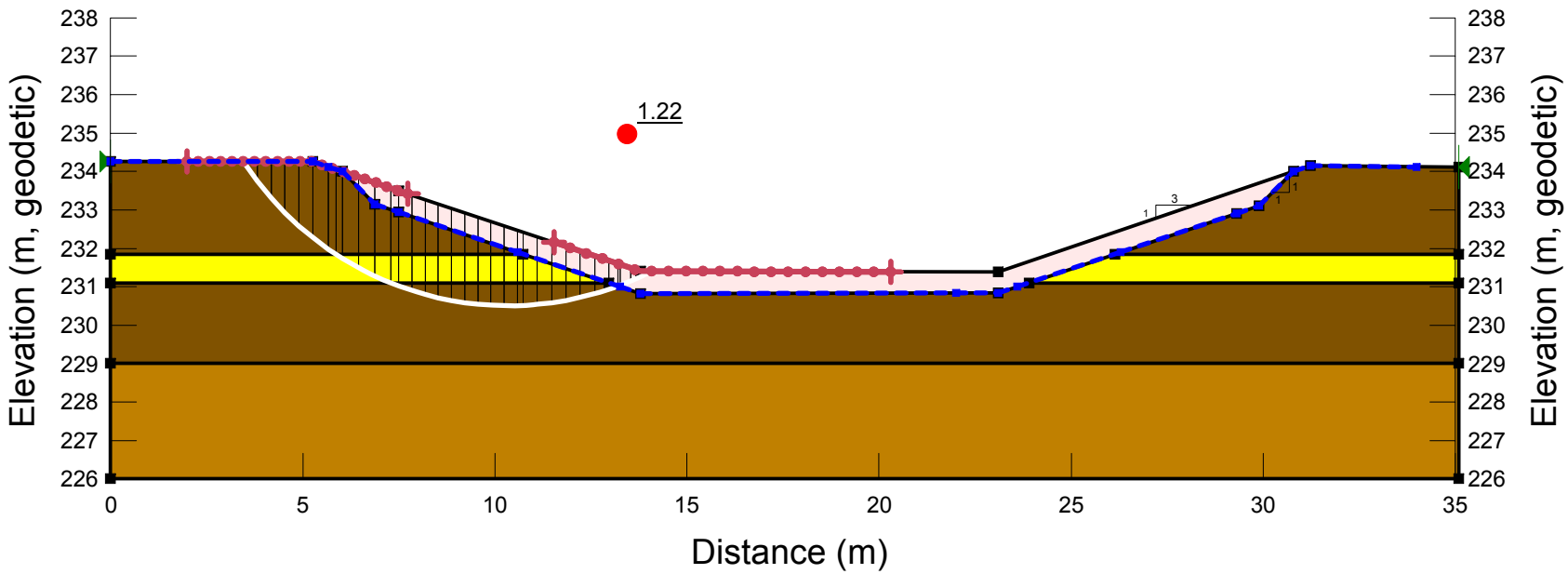


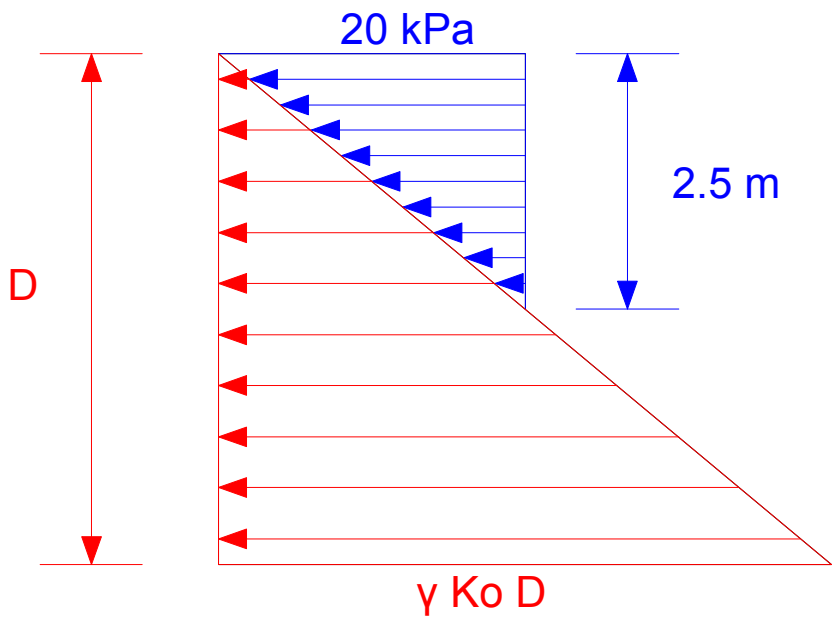
Figure 12 - Culvert Option - Critical Conditions | Date: 03/27/2019

Project Number: 113733340 | Created By: Bonham, Rhett / Checked By: Crilly, Thomas | Scale: NTS

Directory: U:\113733340\0100_analysis\0102_design_calculations\09_geotech\slope stability\feb_2019\dublin_3H1V_culvert_20190327.gsz

Stantec

Appendix H
LATERAL EARTH PRESSURE FIGURE



Design Total Lateral Earth Pressure = Compaction Pressure + "At-Rest" Earth Pressure

Figure 1 - Lateral Earth Pressure Distribution		Date: 9/27/2018
Project Number: 113733340	Project: Dublin Bridge Replacement	
Directory: U:\113733340\0500_reports\0500_final\geotechnical\appendix\Appendix H - Lateral Earth Pressure Figure		

Stantec

Stantec Consulting Ltd.
500–311 Portage Avenue, Winnipeg MB R3B 2B9

August 29, 2018
File: 113733340

Attention: Scott Suderman
Stantec Consulting Ltd.
500-311 Portage Avenue
Winnipeg, MB
R3B 2B9

Dear Scott,

Reference: Geotechnical Investigation for Replacement of the Dublin Avenue Bridge and Associated Regional Street Improvements– Winnipeg, Manitoba

On August 1, 2, and 13, 2018, a total of sixteen core samples were recovered and six testholes were drilled on Dublin Avenue and St. James Street. The purpose of the geotechnical investigation was to determine the thickness of the pavement structure and observe the underlying soil conditions. The testhole locations are shown on the Testhole Location Plans within each attached report. Upon completion of the work, the testholes were backfilled with bentonite and clay cuttings; the top 100 mm were repaired with cold mix asphalt. The testhole locations, pavement structure thickness and laboratory test results are provided on the summary tables within each attached report. Testhole logs, photographs of the core samples and the laboratory test reports are also provided in the attachments.

Each street investigated is reported individually for the reader's convenience. We appreciate the opportunity to assist you on this project. Please contact the undersigned if you have any questions regarding our report.Regards,

Regards,

Stantec Consulting Ltd.

Lee Boughton
Geotechnical Technologist
Phone: (204) 489-5900
Lee.Boughton@stantec.com

German Leal M.Eng., P.Eng.
Associate Geotechnical Engineer
Phone: (204) 928-4005
German.Leal@stantec.com

Attachment: Appendix A – Geotechnical Investigation – Dublin Avenue (St. James Street to Notre Dame Street)
Appendix B – Geotechnical Investigation – St. James Street (Saskatchewan Avenue to Dublin Avenue)

us u:\113733340\0400_field_data\0401_geotech\report\re-formated\let_regional_streets_improvements.docx

August 29, 2018

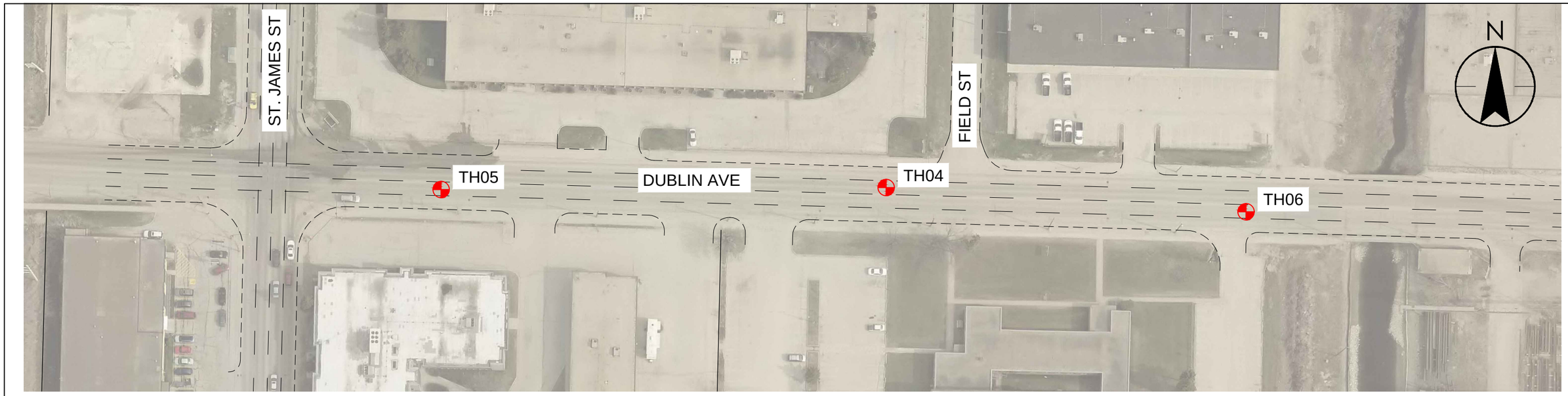
Reference: **Geotechnical Investigation for Replacement of the Dublin Avenue Bridge and Associated Regional Street Improvements– Winnipeg, Manitoba**

Appendix A

Geotechnical Investigation – Dublin Avenue (St. James to Notre Dame Street)

Table 1 – Geotechnical Investigation - Dublin Avenue (St. James to Notre Dame Street)

Testhole ID	Testhole Location	Pavement Surface		Pavement Structure Material		Sample Description	Sample Depth (m)	Moisture Content (%)	Particle Size Analysis				Atterberg Limits		
		Type	Thickness (mm)	Type	Thickness (mm)				Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid Limit	Plastic Limit	Plasticity Index
TH01	Dublin Avenue Eastbound median lane, 126 m east of Orange Street 5.5 m north of south curb	Asphale	135	Crushed Limestone	50	-	-	-	-	-	-	-	-	-	-
		Concrete	180												
TH02	Dublin Avenue Westbound median lane, 78 m west of Midland Street, 5.25 m south of north curb	Asphale	135	Crushed Limestone	1,000	-	-	-	-	-	-	-	-	-	-
		Concrete	215												
TH03	Dublin Avenue Westbound median lane, 8 m east of Orange Street, 5.5 m south of norht curb	Asphale	210	Clay Subgrade	250	Clay	0.9	31	0.0	5.3	24.1	70.6	69	21	48
		Concrete	140												
TH04	Dublin Avenue Westbound median lane, 23 m west of Field Street, 5.5 m south of north curb	Asphale	135	Clay Subgrade	450	-	-	-	-	-	-	-	-	-	-
		Concrete	170												
TH05	Dublin Avenue Eastbound median lane, 40 m east of St. James Street, 5.5 m norht of south curb	Asphale	100	Crushed Limestone	100	Fill (Clay)	0.6	31	0.0	6.2	22.5	71.3	73	23	50
		Concrete	235												
TH06	Dublin Avenue Eastbound median lane, 23 m west of Omands Creek Bridge, 5.5 m norht of south curb	Asphale	160	Crushed Limestone	100	-	-	-	-	-	-	-	-	-	-
		Concrete	170												



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2018/08/28 1:08 PM By: Boughton, Lee

ORIGINAL SHEET - ISO 11x17 - v17.05

2018-08-28
113733340

Stantec

Stantec Consulting Ltd.
Suite 500, 311 Portage Avenue
Winnipeg MB Canada R3B 2B9
Tel. 204.489.5900 Fax. 204.453.9012
www.stantec.com

Legend

TESTHOLE

Notes

Client/Project
CITY OF WINNIPEG
DUBLIN AVENUE REGIONAL STREET IMPROVEMENTS
WINNIPEG, MB

Figure No.
1

Title
TESTHOLE LOCATION PLAN

Figure 2 - TH01 Core

Figure 3 - TH02 Core

Figure 4 - TH03 Core

Figure 5 - TH04 Core

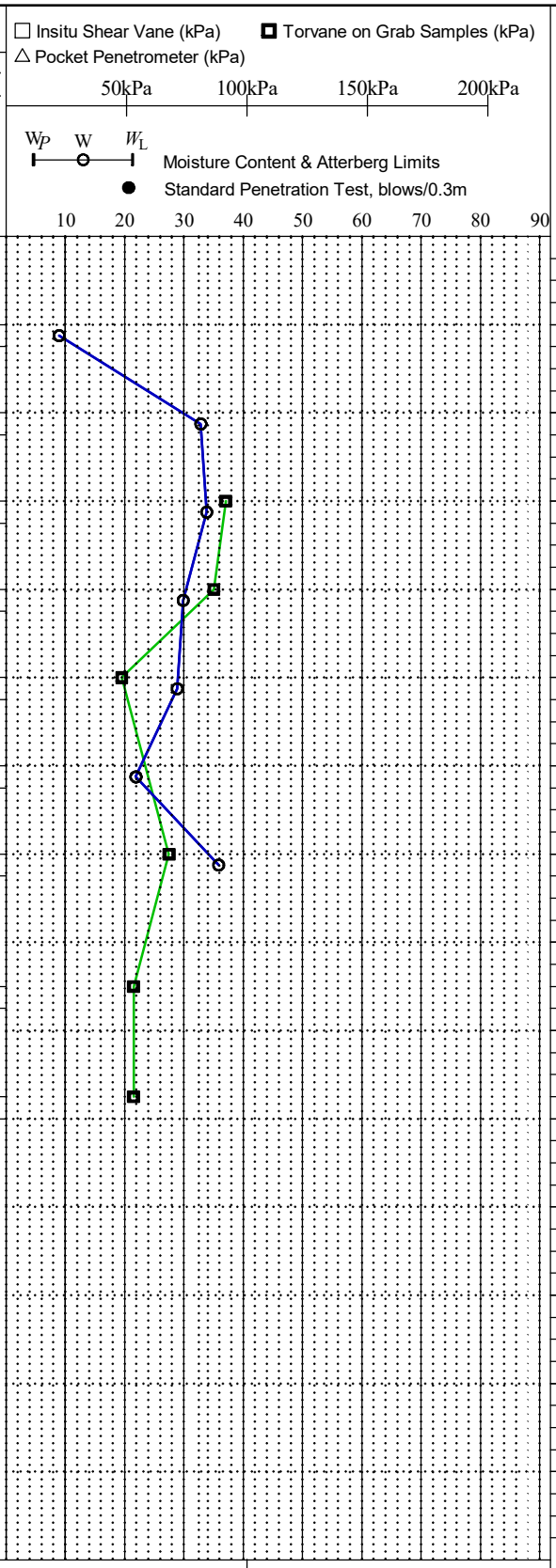
Figure 6 - TH05 Core

Figure 7 - TH06 Core

TH01 TESTHOLE RECORD

CLIENT City of Winnipeg
PROJECT Dublin Avenue Regional Street Improvements
LOCATION Dublin Avenue
DRILLING DATE August 13, 2018
DRILLING CO. Maple Leaf Drilling Ltd.
DATUM UTM
ELEVATION 233.248 m
DRILLING METHOD 125 mm SSA
PROJECT No. 113733340
NORTHING 5530324.384
EASTING 629760.382

DEPTH (m)	SOIL TYPE	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTURE CONTENT (%)
0	AS	ASPHALT			
	CO	CONCRETE			
		3/4 C.L.S.	GS		9
	FL	FILL: stiff, dark grey, fat clay (CH) - silty, trace fine to coarse grained sand - moist	GS		33
1	CH	Stiff, grey, fat CLAY (CH) - silty, trace fine to coarse grained sand - moist	GS		34
			GS		30
	ML	Brown SILT (ML) - some clay, trace fine grained sand - moist	GS		29
			GS		22
2	CH	Stiff, grey, fat CLAY (CH) - silty, trace fine to coarse grained sand - moist	GS		36
			GS		
3			GS		
		End of Testhole • Testhole terminated at a depth of 3.0 m. • No groundwater seepage or soil sloughing was observed during or upon completion of drilling.			
4					



Sample Type: GS - Grab Sample SS - Split Spoon RC - Rock Core
ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite Drill Cuttings Sand Slough

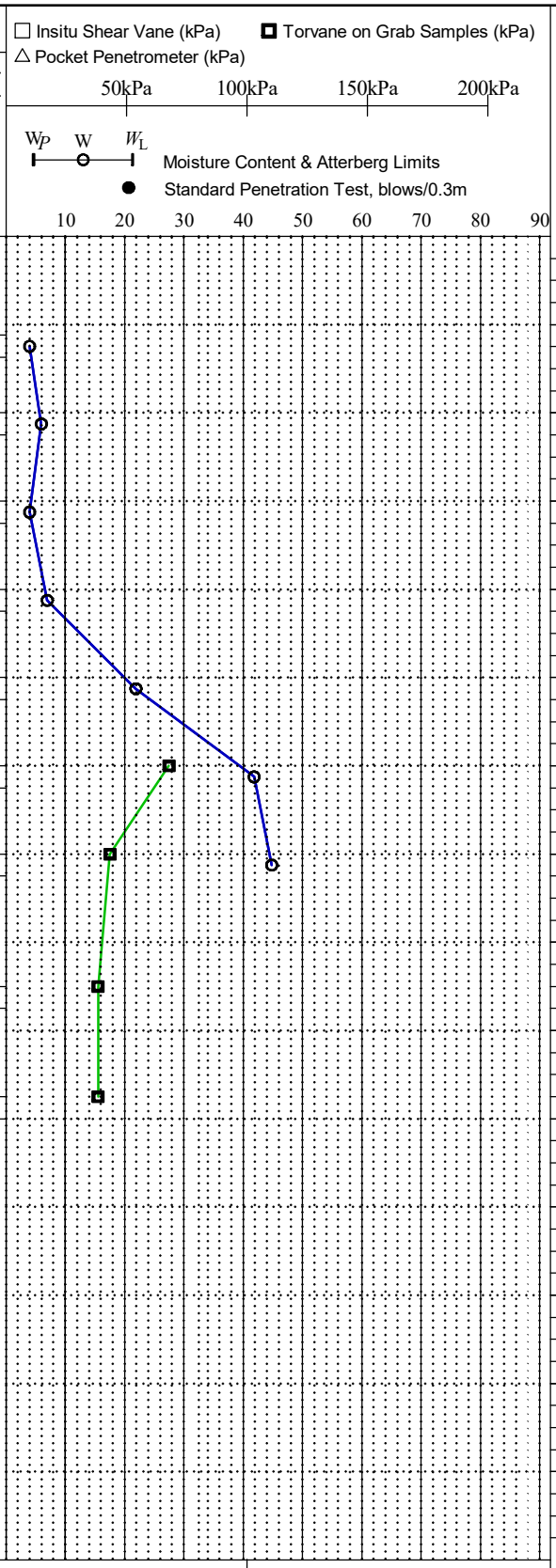
Logged by: Lee Boughton
Reviewed by: German Leal

Stantec

TH02 TESTHOLE RECORD

CLIENT City of Winnipeg
PROJECT Dublin Avenue Regional Street Improvements
LOCATION Dublin Avenue
DRILLING DATE August 13, 2018
DRILLING CO. Maple Leaf Drilling Ltd.
DATUM UTM
ELEVATION 232.859 m
DRILLING METHOD 125 mm SSA
PROJECT No. 113733340
NORTHING 5530324.225
EASTING 629883.389

DEPTH (m)	SOIL TYPE	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTURE CONTENT (%)
0	AS	ASPHALT			
	CO	CONCRETE			
		3/4 C.L.S	GS		4
			GS		6
1			GS		4
			GS		7
	FL	FILL: stiff, dark grey, fat clay (CH) - silty, trace fine to coarse grained sand - moist	GS		22
2	CH	Stiff, grey, fat CLAY (CH) - silty, trace fine to coarse grained sand - moist	GS		42
		- firm below 2.1 m	GS		45
			GS		
3			GS		
4		End of Testhole • Testhole terminated at a depth of 3.0 m. • No groundwater seepage or soil sloughing was observed during or upon completion of drilling.			



Sample Type: GS - Grab Sample SS - Split Spoon RC - Rock Core ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite Drill Cuttings Sand Slough

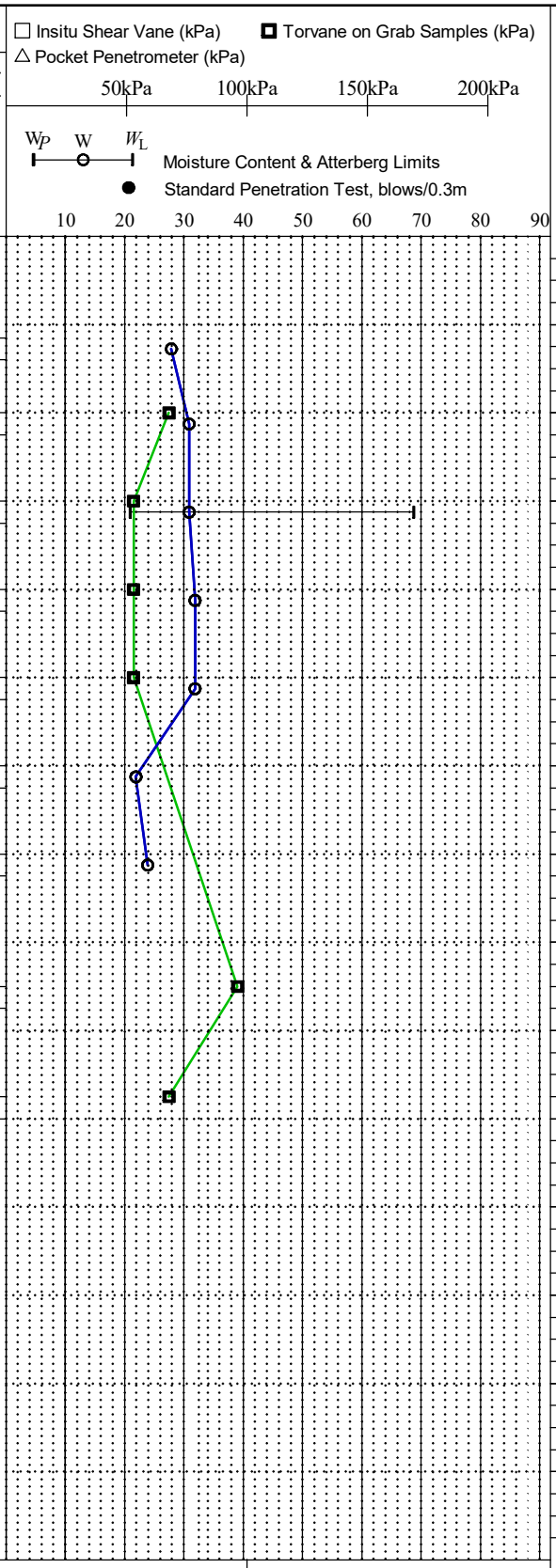
Logged by: Lee Boughton
Reviewed by: German Leal

Stantec

TH03 TESTHOLE RECORD

CLIENT City of Winnipeg
PROJECT Dublin Avenue Regional Street Improvements
LOCATION Dublin Avenue
DRILLING DATE August 13, 2018
DRILLING CO. Maple Leaf Drilling Ltd.
DATUM UTM
ELEVATION 233.347 m
DRILLING METHOD 125 mm SSA
PROJECT No. 113733340
NORTHING 5530330.954
EASTING 629651.467

DEPTH (m)	SOIL TYPE	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTURE CONTENT (%)
0	AS	ASPHALT			
	CO	CONCRETE			
	FL	FILL: stiff, dark grey, fat clay (CH) - silty, trace fine to coarse grained sand - moist	GS		28
			GS		31
1	CH	Stiff, grey, fat CLAY (CH) - silty, trace fine to coarse grained sand - moist Particle Size Analysis Results from SPT @ 0.9 m: - 0% Gravel, 5.3% Sand, 24.1% Silt, 70.6% Clay	GS		31
	ML	Brown, SILT (ML) - trace clay, trace fine grained sand - moist	GS		32
			GS		32
2			GS		22
			GS		24
	CH	Stiff, brown, fat CLAY (CH) - silty, trace fine to coarse grained sand - moist	GS		
3			GS		
		End of Testhole • Testhole terminated at a depth of 3.0 m. • No groundwater seepage or soil sloughing was observed during or upon completion of drilling.			
4					



Sample Type: GS - Grab Sample SS - Split Spoon RC - Rock Core
ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite Drill Cuttings Sand Slough

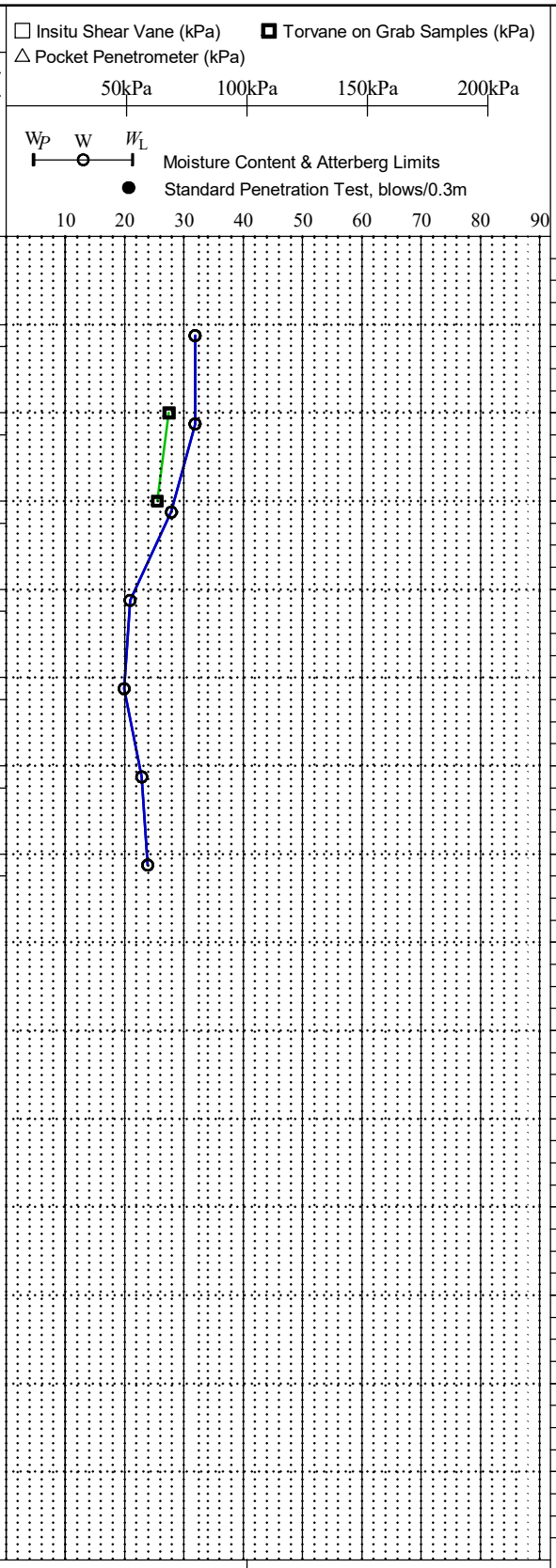
Logged by: Lee Boughton
Reviewed by: German Leal

Stantec

TH04 TESTHOLE RECORD

CLIENT City of Winnipeg
PROJECT Dublin Avenue Regional Street Improvements
LOCATION Dublin Avenue
DRILLING DATE August 13, 2018
DRILLING CO. Maple Leaf Drilling Ltd.
DATUM UTM
ELEVATION 233.446 m
DRILLING METHOD 125 mm SSA
PROJECT No. 113733340
NORTHING 5530336.287
EASTING 629441.497

DEPTH (m)	SOIL TYPE	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTURE CONTENT (%)
0	AS		ASPHALT			
	CO		CONCRETE			
	FL		FILL: stiff, dark grey, fat clay (CH) - silty, trace fine to coarse grained sand - moist	GS		32
				GS		32
	CH		Stiff, grey, fat clay (CH) - silty, trace fine to coarse grained sand - moist	GS		28
1	ML		Brown SILT (ML) - some clay, trace fine grained sand - moist	GS		21
				GS		20
				GS		23
2				GS		24
3			End of Testhole • Testhole terminated at a depth of 3.0 m. • No groundwater seepage or soil sloughing was observed during or upon completion of drilling.			
4						



Sample Type: GS - Grab Sample SS - Split Spoon RC - Rock Core
ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite Drill Cuttings Sand Slough

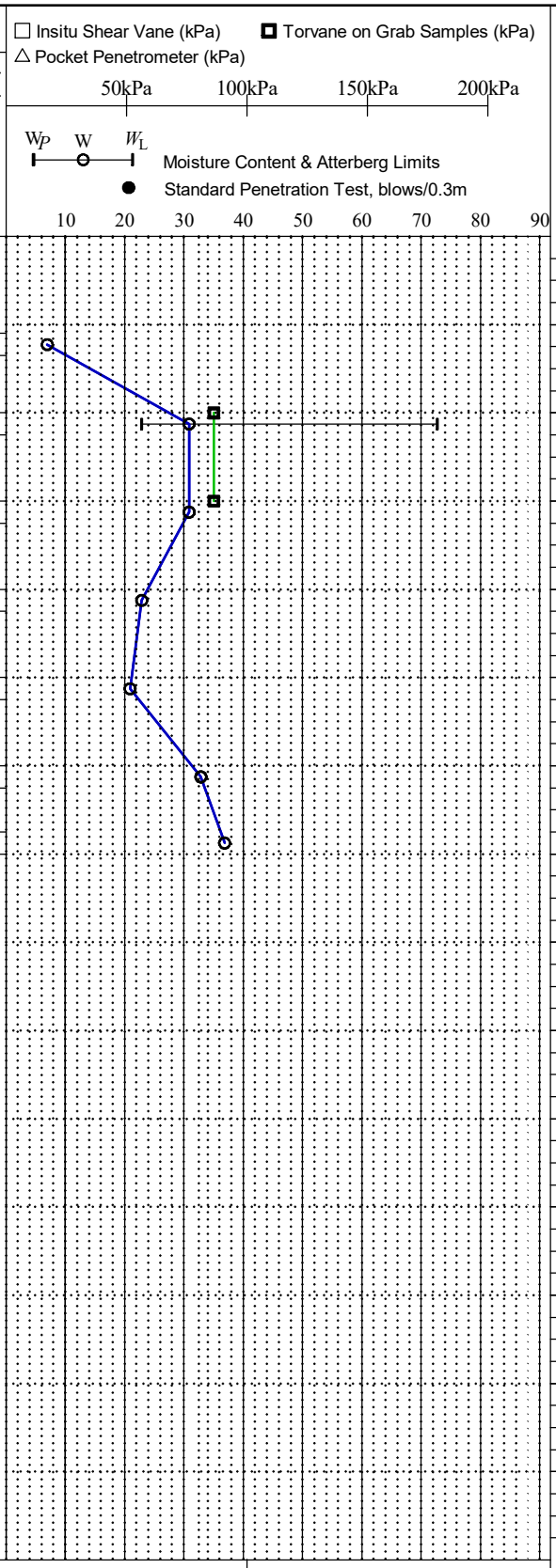
Logged by: Lee Boughton
Reviewed by: German Leal

Stantec

TH05 TESTHOLE RECORD

CLIENT City of Winnipeg
PROJECT Dublin Avenue Regional Street Improvements
LOCATION Dublin Avenue
DRILLING DATE August 13, 2018
DRILLING CO. Maple Leaf Drilling Ltd.
DATUM UTM
ELEVATION 233.589 m
DRILLING METHOD 125 mm SSA
PROJECT No. 113733340
NORTHING 5530336.633
EASTING 629332.758

DEPTH (m)	SOIL TYPE	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTURE CONTENT (%)
0	AS	ASPHALT			
	CO	CONCRETE			
		3/4 C.L.S.	GS		7
	FL	FILL: stiff, dark grey, fat clay (CH) - silty, trace fine to coarse grained sand - moist Particle Size Analysis Results from SPT @ 0.6 m: - 0% Gravel, 6.2% Sand, 22.5% Silt, 71.3% Clay	GS		31
1			GS		31
	ML	Brown SILT (ML) - some clay, trace fine grained sand - moist	GS		23
			GS		21
	CH	Stiff, grey, fat CLAY (CH) - silty, trace fine to coarse grained sand - moist	GS		33
2			GS		37
		End of Testhole • Testhole terminated at a depth of 2.1 m. • No groundwater seepage or soil sloughing was observed during or upon completion of drilling.			
3					
4					



Sample Type: GS - Grab Sample SS - Split Spoon RC - Rock Core
ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite Drill Cuttings Sand Slough

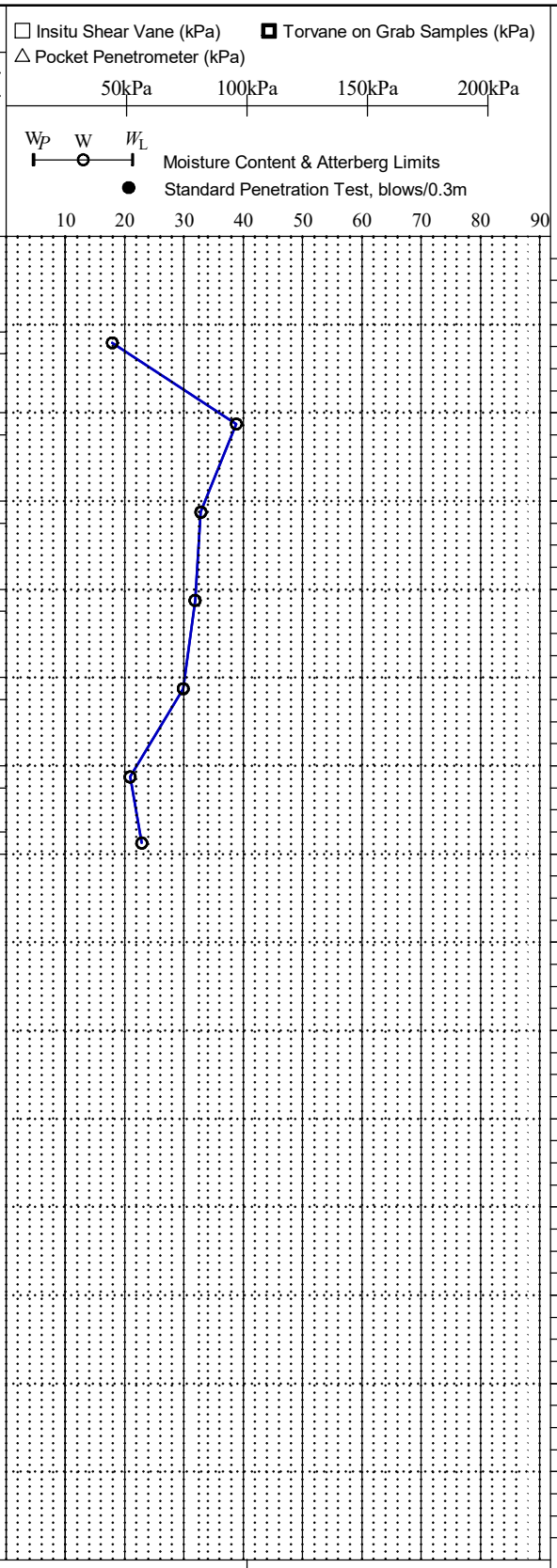
Logged by: Lee Boughton
Reviewed by: German Leal

Stantec

TH06 TESTHOLE RECORD

CLIENT City of Winnipeg
PROJECT Dublin Avenue Regional Street Improvements
LOCATION Dublin Avenue
DRILLING DATE August 13, 2018
DRILLING CO. Maple Leaf Drilling Ltd.
DATUM UTM
ELEVATION 233.771 m
DRILLING METHOD 125 mm SSA
PROJECT No. 113733340
NORTHING 5530330.77
EASTING 629540.89

DEPTH (m)	SOIL TYPE	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTURE CONTENT (%)
0	AS		ASPHALT			
	CO		CONCRETE			
			3/4 C.L.S.	GS		18
	FL		FILL: stiff, dark grey, fat clay (CH) - silty, trace fine to coarse grained sand - moist	GS		39
1				GS		33
	ML		Brown, SILT (ML) - some clay, trace fine grained sand - moist	GS		32
				GS		30
				GS		21
2				GS		23
			End of Testhole • Testhole terminated at a depth of 2.1 m. • No groundwater seepage or soil sloughing was observed during or upon completion of drilling.			
3						
4						



Sample Type: GS - Grab Sample SS - Split Spoon RC - Rock Core
ST - Shelby Tube PT - Piston Tube VT - Shear Vane Test

Piezometer Backfill Type: Bentonite Drill Cuttings Sand Slough

Logged by: Lee Boughton
Reviewed by: German Leal

Stantec

LABORATORY
199 Henlow Bay
Winnipeg MB R3Y 1G4
Tel: (204) 488-6999

PARTICLE SIZE ANALYSIS
ASTM D422

Stantec Consulting Ltd.
500 - 311 Portage Avenue
Winnipeg, Manitoba
R3B 2B9

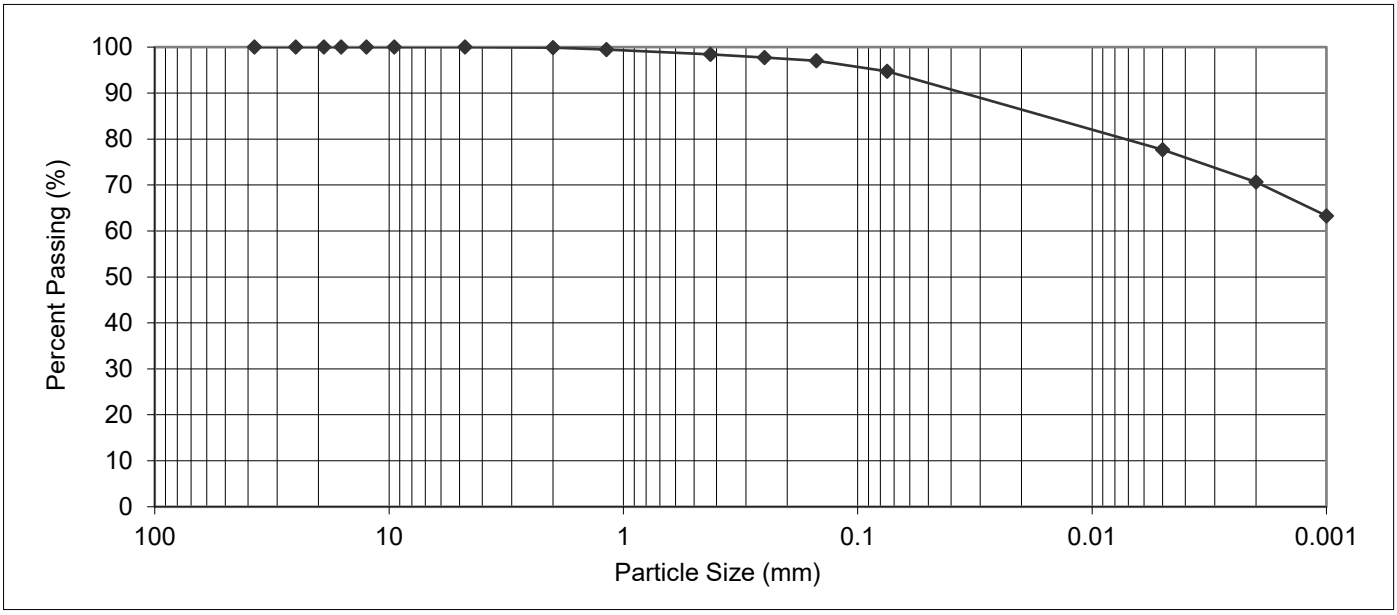
PROJECT: Dublin Bridge Replacement

Attention:

PROJECT NO.: 113733340

SAMPLED BY: Lee Boughton,
SAMPLE ID: TH03 @ 3'

DATE RECEIVED: August 17, 2018
TESTED BY: Nestor Abarca, C.Tech.



PARTICLE SIZE	PERCENT PASSING
37.50 mm	100.0
25.00 mm	100.0
19.00 mm	100.0
16.00 mm	100.0
12.50 mm	100.0
9.50 mm	100.0
4.75 mm	100.0
2.00 mm	99.9

PARTICLE SIZE	PERCENT PASSING
1.18 mm	99.5
0.425 mm	98.4
0.250 mm	97.7
0.150 mm	97.0
0.075 mm	94.7
0.005 mm	77.7
0.002 mm	70.6
0.001 mm	63.3

Gravel, % 75 to 4.75 mm	Sand, %			Silt, % <0.075 to 0.002 mm	Clay, % <0.002 mm	Colloids, % < 0.001 mm
	Coarse <4.75 to 2.0 mm	Medium <2.0 to 0.425 mm	Fine <0.425 to 0.075 mm			
0.0	0.1	1.5	3.7	24.1	70.6	63.3

REPORT DATE: August 24, 2018

REVIEWED BY: German Leal, B.Sc., P. Eng.

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request. The data presented above is for the sole use of the client stipulated above. Stantec is not responsible, nor can be held liable, for the use of this report by any other party, with or without the knowledge of Stantec.

LABORATORY
199 Henlow Bay
Winnipeg MB R3Y 1G4
Tel: (204) 488-6999

PARTICLE SIZE ANALYSIS
ASTM D422

Stantec Consulting Ltd.
500 - 311 Portage Avenue
Winnipeg, Manitoba
R3B 2B9

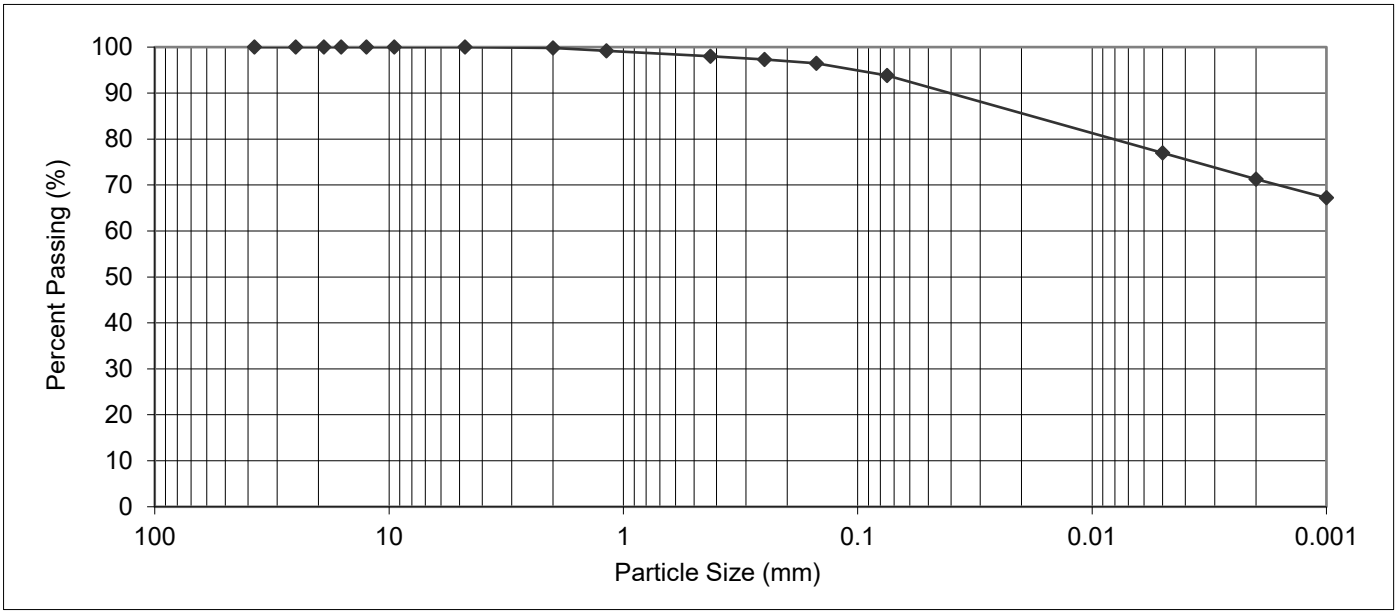
PROJECT: Dublin Bridge Replacement

Attention:

PROJECT NO.: 113733340

SAMPLED BY: Lee Boughton,
SAMPLE ID: TH05 @ 2'

DATE RECEIVED: August 17, 2018
TESTED BY: Nestor Abarca, C.Tech.



PARTICLE SIZE	PERCENT PASSING
37.50 mm	100.0
25.00 mm	100.0
19.00 mm	100.0
16.00 mm	100.0
12.50 mm	100.0
9.50 mm	100.0
4.75 mm	100.0
2.00 mm	99.8

PARTICLE SIZE	PERCENT PASSING
1.18 mm	99.2
0.425 mm	98.0
0.250 mm	97.3
0.150 mm	96.5
0.075 mm	93.8
0.005 mm	76.9
0.002 mm	71.3
0.001 mm	67.2

Gravel, % 75 to 4.75 mm	Sand, %			Silt, % <0.075 to 0.002 mm	Clay, % <0.002 mm	Colloids, % < 0.001 mm
	Coarse <4.75 to 2.0 mm	Medium <2.0 to 0.425 mm	Fine <0.425 to 0.075 mm			
0.0	0.2	1.8	4.2	22.5	71.3	67.2

REPORT DATE: August 24, 2018

REVIEWED BY: German Leal, B.Sc., P. Eng.

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Atterberg Limits
ASTM D4318
Method A- Multi-Point

Client: Stantec Consulting Ltd.
Project Name: Dublin Bridge Replacement

Project No: 113733340
Date Received: August 17, 2018
Date Tested: August 23, 2018
Tested By: Nestor Abarca, C. Tech.

LABORATORY
199 Henlow Bay

Winnipeg, Manitoba
Canada R3Y 1G4
Tel: (204) 488-6999

Sample : TH03 @ 3'

LIQUID LIMIT

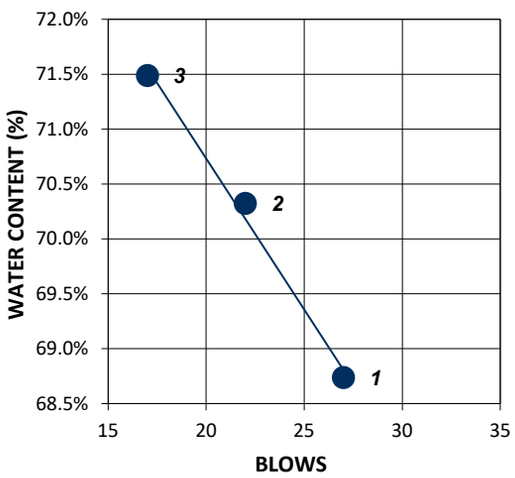
Trial	1	2	3
No. of Blows	27	22	17
Tare No.	154	202	204
Wt. Sa. (wet+tare)(g)	41	41	40
Wt. Sa. (dry+tare)(g)	32	32	31
Wt. Tare (g)	19	19	18
Wt. Dry Soil (g)	13.4	13.3	12.5
Wt. Water (g)	9.2	9.4	8.9
Water Content (%)	68.7%	70.3%	71.5%

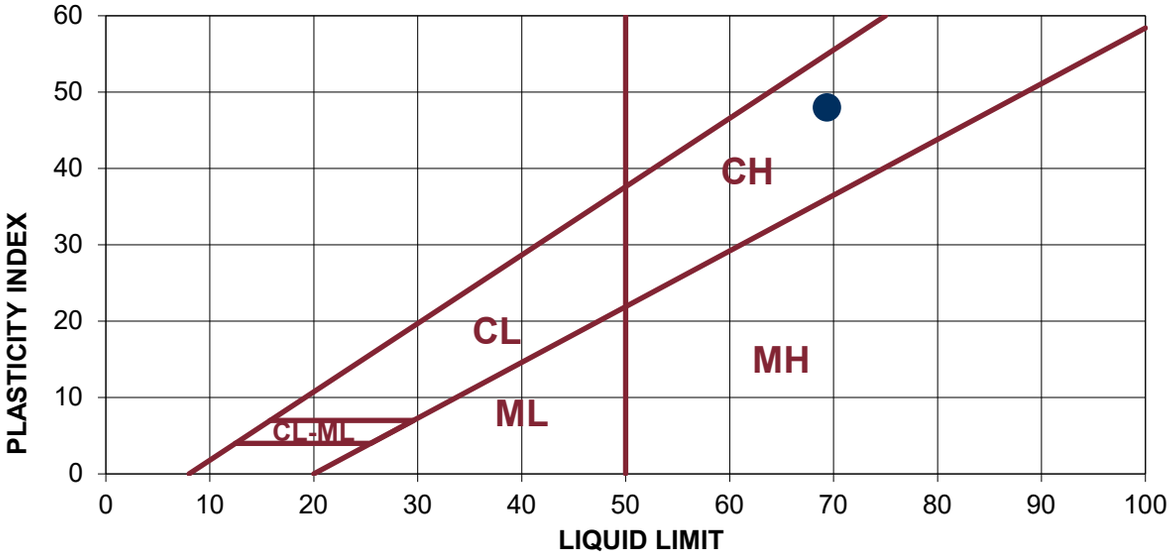
PLASTIC LIMIT

Trial	1	2
Tare No.	296	303
Wt. Sa. (wet+tare)(g)	32.41	31.33
Wt. Sa. (dry+tare)(g)	30.28	29.47
Wt. Tare (g)	20.05	20.58
Wt. Dry Soil (g)	10.2	8.9
Wt. Water (g)	2.1	1.9
Water Content (%)	20.8%	20.9%

RESULTS

LL	69
PL	21
PI	48
Natural MC (%)	
30.9%	





Reviewed By: German Leal, B.Sc., P.Eng.

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Atterberg Limits
ASTM D4318
Method A- Multi-Point

Client: Stantec Consulting Ltd.
Project Name: Dublin Bridge Replacement

Project No: 113733340
Date Received: August 17, 2018
Date Tested: August 23, 2018
Tested By: Nestor Abarca, C. Tech.

LABORATORY
199 Henlow Bay

Winnipeg, Manitoba
Canada R3Y 1G4
Tel: (204) 488-6999

Sample : TH05 @ 2'

LIQUID LIMIT

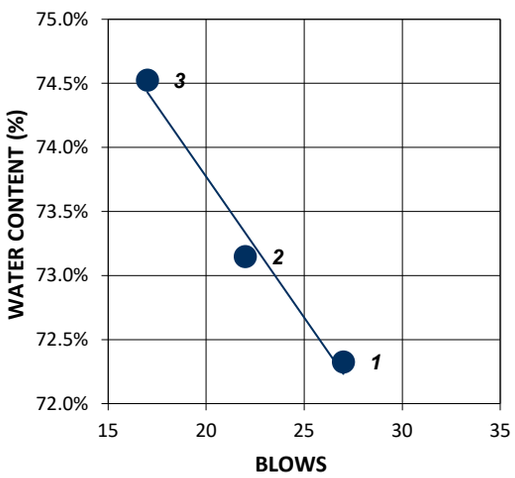
Trial	1	2	3
No. of Blows	27	22	17
Tare No.	151	256	285
Wt. Sa. (wet+tare)(g)	39	42	42
Wt. Sa. (dry+tare)(g)	31	33	33
Wt. Tare (g)	19	20	21
Wt. Dry Soil (g)	11.5	12.6	12.1
Wt. Water (g)	8.3	9.2	9.0
Water Content (%)	72.3%	73.1%	74.5%

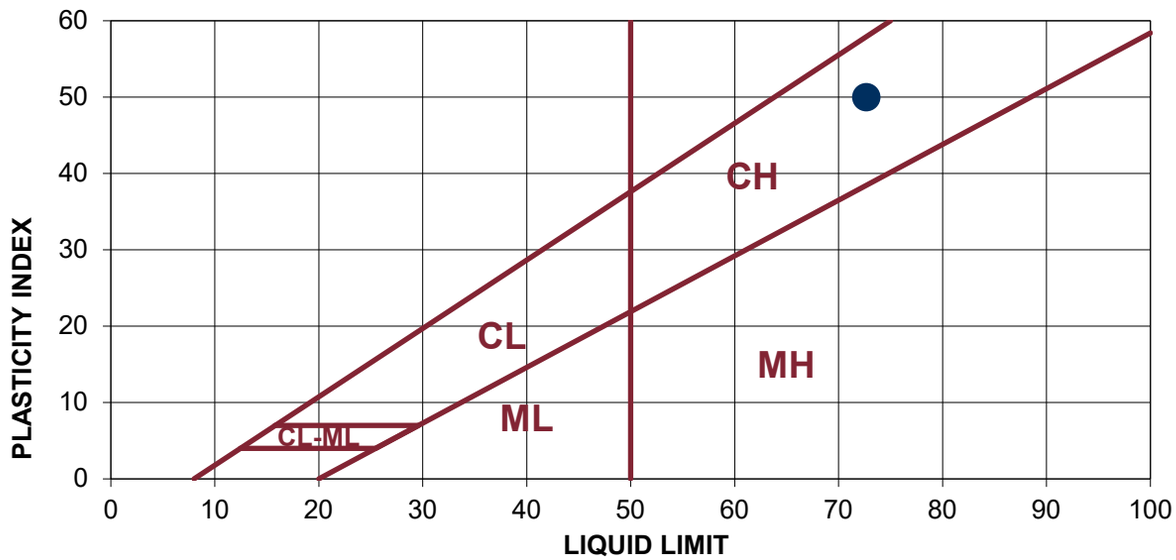
PLASTIC LIMIT

Trial	1	2
Tare No.	304	317
Wt. Sa. (wet+tare)(g)	31.21	32.46
Wt. Sa. (dry+tare)(g)	29.11	30.16
Wt. Tare (g)	19.99	20.17
Wt. Dry Soil (g)	9.1	10.0
Wt. Water (g)	2.1	2.3
Water Content (%)	23.0%	23.0%

RESULTS

LL	73
PL	23
PI	50
Natural MC (%)	
30.6%	





Reviewed By: German Leal, B.Sc., P.Eng.

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request. The data presented above is for the sole use of the client stipulated above. STANTEC is not responsible, nor can be held liable, for the use of this report by any other party, with or without the knowledge of STANTEC.

August 29, 2018

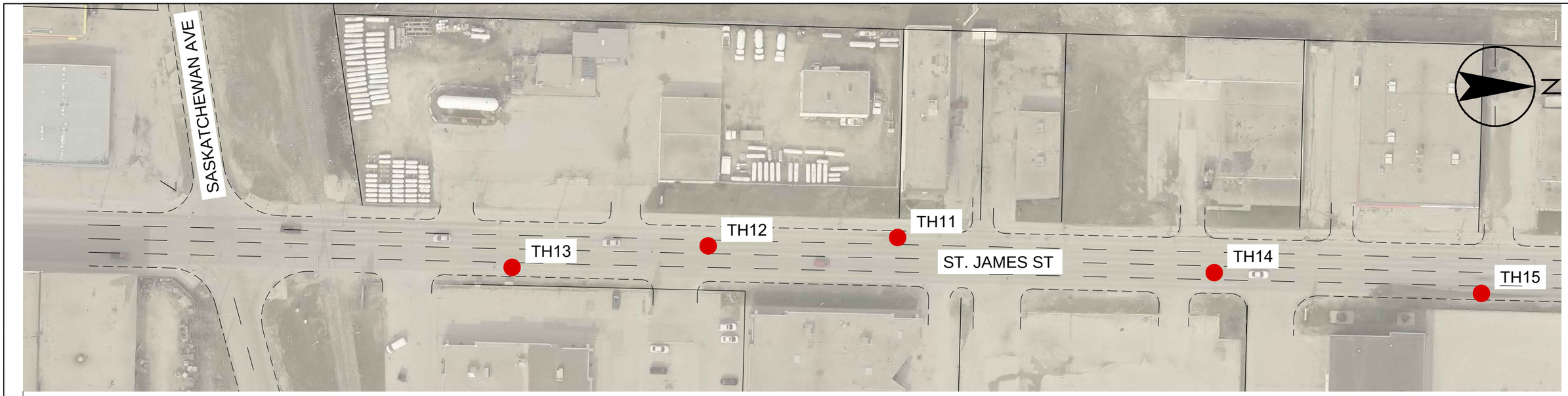
Reference: **Geotechnical Investigation for Replacement of the Dublin Avenue Bridge and Associated Regional Street Improvements– Winnipeg, Manitoba**

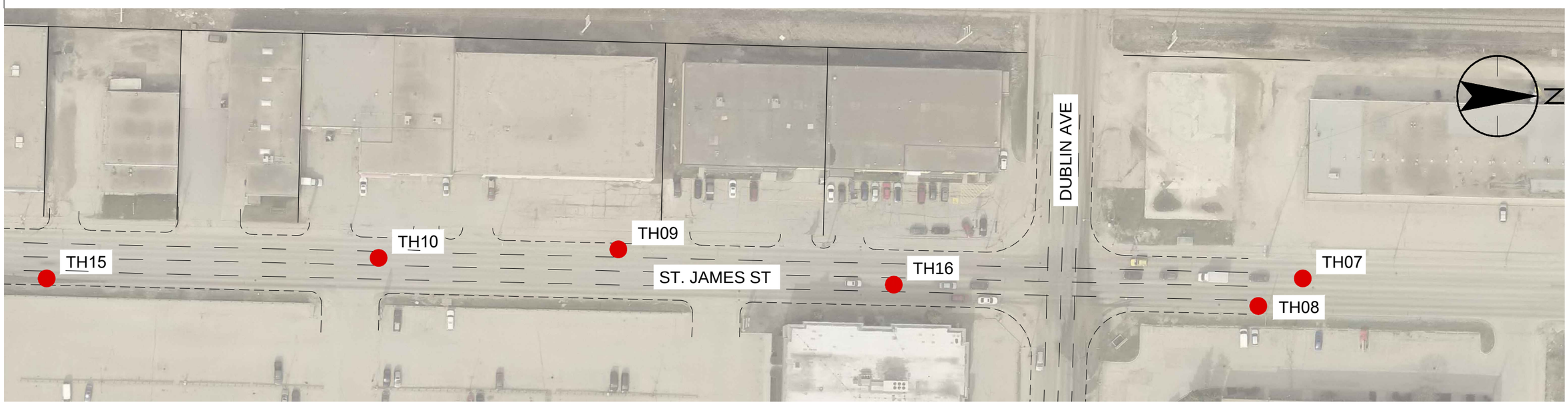
Appendix B

Geotechnical Investigation – St. James Street (Saskatchewan Avenue to Dublin Avenue)

Table 1 - Geotechnical Investigation - St. James Street (Saskatchewan Avenue to Dublin Avenue)

Testhole ID	Testhole Location	Pavement Surface		Pavement Structure Material		Sample Description	Sample Depth (m)	Moisture Content (%)	Particle Size Analysis				Atterberg Limits		
		Type	Thickness (mm)	Type	Thickness (mm)				Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid Limit	Plastic Limit	Plasticity Index
TH07	St. James Street Southbound median lane, 50 m north of Dublin Avenue, 5 m east of west curb	Asphale	35	Crushed Limestone	-	-	-	-	-	-	-	-	-	-	-
		Concrete	235												
TH08	St. James Street North bound curb lane, 43 m north of Dublin Avenue, 2 m west of east curb	Asphale	25	Crushed Limestone	-	-	-	-	-	-	-	-	-	-	-
		Concrete	225												
TH09	St. James Street Southbound curb lane, 106 m south of Dublin Avenue, 2.25 m east of west curb	Asphale	55	Crush Limestone	-	-	-	-	-	-	-	-	-	-	-
		Concrete	210												
TH10	St. James Street Southbound median lane, 167 m south of Dublin Avenue, 5 m east of west curb	Asphale	65	Crushed Limestone	-	-	-	-	-	-	-	-	-	-	-
		Concrete	250												
TH11	St James Street Southbound curb lane, 181 m north of Saskatchean Avenue, 2 m east of west curb	Asphale	50	Crushed Limestone	-	-	-	-	-	-	-	-	-	-	-
		Concrete	215												
TH12	St. James Street Southbound median lane, 124 m north of Saskatchewan Avenue, 5.5 m east of west curb	Asphale	60	Crushed Limestone	-	-	-	-	-	-	-	-	-	-	-
		Concrete	230												
TH13	St. James Street Northbound curb lane, 80 m north of Saskatchewan Avenue, 1.75 m west of east curb	Asphale	35	Crushed Limestone	-	-	-	-	-	-	-	-	-	-	-
		Concrete	240												
TH14	St. James Street Northbound median lane, 264 m south of Dublin Avenue, 5.25 m west of east curb	Asphale	65	Crushed Limestone	-	-	-	-	-	-	-	-	-	-	-
		Concrete	235												
TH15	St. James Northbound curb lane, 252 m south of Dublin Avenue, 2 m west of east curb	Asphale	65	Crushed Limestone	-	-	-	-	-	-	-	-	-	-	-
		Concrete	210												
TH16	St. James Street Northbound median lane, 40 m south of Dublin Avenue, 5 m west of east curb	Asphale	55	Crushed Limestone	-	-	-	-	-	-	-	-	-	-	-
		Concrete	200												





ORIGINAL SHEET - ISO 11x17 - v17.05

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2018/08/28 1:17 PM By: Boughton, Lee

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Legend

TESTHOLE (CORING ONLY)

Notes

Client/Project
CITY OF WINNIPEG
ST. JAMES STREET REGIONAL STREET IMPROVEMENTS
WINNIPEG, MB

Figure No.
8

Title
TESTHOLE LOCATION PLAN

Figure 9 - TH07 Core

Figure 10 - TH08 Core

Figure 11 - TH09 Core

Figure 12 - TH10 Core

Figure 13 - TH11 Core

Figure 14 - TH12 Core

Figure 15 - TH13 Core

Figure 16 - TH14 Core

Figure 17 - TH15 Core

Figure 18 - TH16 Core